

FIELD AND LABORATORY STUDY OF MODULUS OF RUPTURE AND
PERMEABILITY OF STRUCTURAL CONCRETES IN FLORIDA

By

PRASIT SOONGSWANG

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Abstract of Dissertation Presented to the Graduate School
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FIELD AND LABORATORY STUDY OF MODULUS OF RUPTURE AND
PERMEABILITY OF STRUCTURAL CONCRETES IN FLORIDA

By

Prasit Soongswang

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Chairman: Mang Tia
Cochairman: David G. Bloomquist
Major Department: Civil Engineering

The durability of concrete is greatly affected by the modulus of rupture and permeability of the concrete. However, data on these concrete properties are lacking at present, especially for Florida concretes. An extensive laboratory study was conducted to evaluate the effects of various mix parameters and curing conditions on these two properties of structural concretes used in Florida, and the relationship of other concrete properties, namely compressive strength, splitting tensile strength, modulus of elasticity and Schmidt hammer rebound number, to these two properties. An efficient laboratory test set-up for performing forty permeability tests simultaneously was developed and used in this study. In addition, concrete samples obtained from six concrete projects in Florida were tested to evaluate the differences between the in-service concrete and the laboratory-cured concrete.

The scope of the laboratory study covered three aggregate types (a Florida porous limestone, a river gravel, and an Alabama dense limestone), three cement types (Types I, II and III), three water/cement-ratios (0.45, 0.38 and 0.33), three maximum aggregate sizes ($\frac{3}{8}$, $\frac{3}{4}$ and 1 inch), two pozzolans (fly ash and silica fume) and six curing conditions (steam curing, air curing, curing compound and three moist curing conditions). A total of 58 batches of concrete were prepared and over 2000 concrete specimens were tested.

Analyses of variance and regression analyses were performed on the results to determine the significance of test parameters and to establish relationships. The results of the study indicated that water/cement-ratio and curing condition are the major factors affecting the properties of concrete. The replacement of cement by 20% Class F fly ash produced about the same modulus of rupture but lower permeability than the control concrete at 28-day and 90-day moist curing. The silica fume concrete reaches its minimum permeability at a silica fume addition of around 10% by weight of cement while the modulus of rupture reaches its maximum at 20% addition. Empirical relationships between different concrete properties were developed and were found to be very close to those in the ACI codes.

An optimum mix design, which minimizes permeability, maximizes strengths and is suitable for use in practice, was recommended as a result of the study.

CHAPTER 1 INTRODUCTION

1.1 Background and Research Needs

In recent years, the premature deterioration of concrete structures has become a major concern for the Florida Department of Transportation (FDOT). The FDOT has to spend millions of dollars annually for the repair and replacement of deteriorated concrete structures. Corrosion of the steel rebar is one of the main factors causing the damage to concrete structures. The formation of corrosion products induces internal stress, which can cause cracking and spalling of concrete that covers the reinforcing steel. Another possible factor that can cause the deterioration of concrete structures is the formation of cracks due to the excessive tensile stress from the applied loads.

In order to overcome this problem of premature deterioration, two properties of structural concrete, namely modulus of rupture and permeability, need to be studied. The reason for this is because the long-term performance of concrete structures is influenced greatly by the modulus of rupture and permeability of the concrete. The modulus of rupture determines the maximum flexural stress that concrete can sustain before cracking and thus affects the resistance of the structure to applied loads. The permeability of concrete controls the rate of flow of water, which can cause disruption to the concrete upon freezing. The permeability of concrete also controls the rate of flow of chemicals such as chloride ions, which can reduce the pH of the concrete and

increase the rate of corrosion of the steel rebars in the concrete structure. This study was performed to obtain a better understanding of the factors that affect these two properties of concrete. From this study, the durability of structural concretes in Florida can be adequately assessed and controlled.

There currently does not exist any widely used standard procedure for determining the permeability of concrete. In order to perform an extensive study on the permeability of Florida concrete, an efficient test set-up for reliably measuring the permeability of concrete was needed.

1.2 Objectives of the Study

The objectives of this study are

1. to develop an efficient laboratory test set-up for measuring the permeability of concrete,
2. to determine the magnitudes of modulus of rupture and water permeability of Florida concrete,
3. to determine the effects of aggregate type, cement type, maximum aggregate size, fly ash, silica fume, and curing condition on the modulus of rupture and water permeability of Florida concrete,
4. to determine the relationships of other concrete properties (such as Schmidt hammer rebound number, compressive strength, splitting tensile strength and modulus of elasticity) to modulus of rupture and permeability of Florida concrete,
5. to compare the modulus of rupture and permeability of in-service concretes with those of laboratory-cured concrete, and

6. to develop an optimum concrete mix which minimizes permeability, maximizes rupture strength and is suitable for use in practice.

1.3 Summary of the Following Chapters

Chapter 2 presents a literature review on the factors affecting the permeability and modulus of rupture of concrete, and the test methods used to measure these two properties. Chapter 3 describes the development of the laboratory permeability test set-up which was developed and used in the laboratory testing program of this study. The design of experiment and testing procedures for the laboratory study are presented in Chapter 4. The materials used in the laboratory study are documented in Chapter 5. Chapter 6 presents the method of analysis of data. Results of modulus of rupture, compressive strength, splitting tensile strength and permeability tests are presented in Chapters 7, 8, 9 and 10, respectively. Chapter 11 presents the recommended mix design. Development of prediction equations for concrete properties is presented in Chapter 12. The results of the field testing program are presented in Chapter 13. Finally, Chapter 14 presents the conclusions and recommendations obtained from this study.

CHAPTER 2 LITERATURE REVIEW

2.1 Introduction

The permeability of concrete plays a very important role in influencing the durability of a concrete structure. First, it controls the rate of flow of water, which can cause disruption to the concrete upon freezing. Secondly, it controls the rate of flow of chemicals, such as chloride ions, which can reduce the pH of the concrete and increase the rate of corrosion of the steel reinforcement in concrete structures. Therefore, the permeability of concrete to water is a property of interest to nearly all designers of concrete structures [1].

The modulus of rupture or flexural strength of concrete has a great effect on its resistance to loads. Its value is often determined from flexural tests on plain test beams. This method has been used primarily for control of concrete mixes on structures where the design is based upon the modulus of rupture [2]. This chapter presents an overview of prior permeability research, factors affecting this parameter, and the modulus of rupture of concrete.

2.2 Permeability of Concrete

2.2.1 Basics

Permeability is the ease with which liquids or gases can travel through the concrete. This is an important property affecting the watertightness of liquid-retaining structures, resistance to chemical

attack, development of hydrostatic pressure in the interior of the concrete mass, and ingress of moisture into the concrete wall which affects its thermal insulation properties [3,4].

Neville [4] pointed out that movement of water through a concrete of certain thickness can be caused not only by a head of water but also by a humidity differential between the two sides of the concrete or by osmotic effects.

We should distinguish between permeability and absorption. Absorption is normally measured by drying a specimen to a constant weight, immersing it in water and measuring the increase in weight as a percentage of dry weight, so that absorption represents the maximum holding capacity of water in relation to a prescribed condition of dryness. Permeability, on the other hand, is concerned with flow through the concrete [4].

The flow of water through concrete is essentially a flow through a porous body but the problem is far from simple. Both the cement paste and the aggregate contain pores, and, in addition, the concrete as a whole may contain voids due to incomplete compaction or to bleeding. These voids may occupy a fraction of 1% to 10% of the volume of the concrete. The latter figure represents a highly honey-combed concrete of very low strength which can be avoided by proper construction techniques. Since aggregate particles are enveloped by the cement paste in fully compacted concrete, it is the permeability of the paste that has the greatest effect on the permeability of concrete. The pores in the cement paste are gel pores and capillary pores. The gel pores constitute about 28% of the paste volume, and the capillary pores between 0%

and 40%, depending on the water/cement-ratio (w/c-ratio) and the degree of hydration [4,5].

The permeability of concrete is not a simple function of its porosity, but depends also on the size, distribution, and continuity of the pores. Thus, although the cement gel has a porosity of 28%, its permeability is only about 2.75×10^{-14} inches/sec. This is due to the extremely fine texture of hardened cement paste. The pores and the solid particles are very small and numerous, while in rocks the pores, though fewer in number, are much larger and lead to a higher permeability. The water can flow more easily through the capillary pores than through the much smaller gel pores. The cement paste as a whole is 20 to 100 times more permeable than the gel itself. It follows that the permeability of cement paste is controlled by its capillary porosity. Since the capillary porosity of cement paste decreases with age, permeability also decreases [4,5].

2.2.2 Test Methods

There currently does not exist any widely used standard procedure for determining the water permeability of concrete [3]. A standard test method for water permeability of concrete is now being studied by the American Society for Testing and Materials (ASTM). However, quite a few different methods for measuring permeability have been used in the past. For example, McMillan and Lyse [6] have studied the permeability of concrete at the Research Laboratory of the Portland Cement Association. The specimens used in this study were 6 inches in diameter and 2 inches in height. A layer $1/8$ -inch thick was removed from the top and bottom surface of each specimen. The sidewalls of the specimens were sealed with paraffin and resin. Water was forced into the top surface of the

specimen with a pressure of 80 psi. The flow rate of water under pressure was measured by the outflow method.

Norton and Pletta [7] have studied the permeability of gravel concrete in the Mechanics Department at the University of Wisconsin in the hope of determining the relationship between permeability and w/c-ratio, and also the relationship between permeability and other properties such as strength, consistency, absorption, cement-void ratio and grading of aggregate. The specimens used in this study were 9.5 inches in diameter and 6 inches in height. The outer film of concrete on the top and bottom of the specimens was removed with a wire brush. The sidewalls of the specimens were sealed with asphalt. The flow rate of water under pressure was measured by the inflow method by means of a water column. The pressure on the top of the water column was maintained at 40 psi except in some of the later tests where 100 psi was used.

Ruettgers et al. [8] have studied the permeability of mass concrete with particular reference to Boulder Dam in Colorado. The permeability investigations in this project were made primarily to furnish data for making final decisions on the type of construction and the mix or mixes to be used. The large aggregate size (up to 9 inches) was used in this study. The apparatus was designed for testing two different sizes of specimens at the same time and for measuring both inflow and outflow. One specimen is 6 inches in diameter and 12 inches maximum in height. The other specimen is 18 inches in diameter and 24 inches maximum in height. A layer of $1/8$ inch of mortar was chipped from the top and bottom surfaces of the specimens before testing. The sidewalls of the specimens were sealed with a commercial pipe-joint asphalt compound.

The flow rate of water under pressure through the concrete specimen was measured after a steady flow condition had been reached by both inflow and outflow methods. A pressure of 400 psi was used in testing the specimens. The tests were usually continued until the flow rate of inflow and outflow were in close agreement, generally from 200 to 500 hours after commencement of test. A tangent drawn to the cumulative inflow curve at a point corresponding to 250 hours plus one-half the time to visible outflow determined the flow rate for computing the permeability coefficient.

Tyler and Erlin [1] have studied a proposed simple test method for determining the permeability of concrete. The apparatus was modified from the high pressure apparatus developed by J. D. Lindsay of the Illinois Department of Highways, originally intended for determining air content of hardened concrete. Concrete specimens 6 inches in diameter and 12 inches in height were used in this study. Water was forced into the concrete specimen by pressure up to 500 psi. The amount of water forced into the specimens was measured by the inflow method.

Von der Meulen and Van Dijk [9] have studied the permeability of concrete at the South African National Building Research Institute. Concrete specimens 3 inches in diameter and 1 inch in height were used in this study. The sidewall of the specimen was sealed with epoxy. The amount of water passing through the concrete specimen was measured by the outflow method.

Janssen [11] has developed the permeameter, which can measure the actual permeability from field or laboratory samples. He claimed that his apparatus is simple and inexpensive. The concrete sample was 3 inches in diameter and 3 inches long. A brass sample ring 3 1/2 inches

in diameter by $3 \frac{1}{8}$ inches long was used as a sample cell. The side-walls of the concrete sample and the inside of the brass sample ring were coated with Dow-Corning Concrete Sealer and allowed to cure overnight. Additional sealer was then spread around the sample and the sample was inserted into the brass sample ring, and the sealer was allowed to cure for about 24 hours. The sample was vacuum saturated for 24 hours. The de-aired water under pressure of 40 psi was forced into the concrete sample. The water flow through the sample was collected from the bottom side of the sample.

Bisaillon and Malhotra [12] have modified the permeability apparatus which was originally developed by Hope and Malhotra. Concrete specimens 6 inches in diameter and 5 inches in height were prepared and used. The specimens were allowed to dry for one day at room temperature after removal from the moist room. Epoxy resin incorporating mineral fillers and fine sand was used to seal the circumference of the specimens. The pressure of 500 psi was used to force the deaerated water into the concrete specimen. The flow rate of water under pressure through the concrete specimen was measured by an outflow method.

Murata [13] has studied a method in which the permeability of concrete is measured by means of a diffusion coefficient obtained from the relation between the average depth of penetration of water forced under pressure into concrete and hydrostatic pressure, and the period of time the pressure was applied. Specimens 6 inches in diameter and 6 inches in height were used. The top and bottom surfaces of the specimens were finished by grinding. The sidewall of the specimens was sealed with a paraffin and resin. Water was forced into the concrete specimens with a pressure of 70 to 280 psi (5 to 20 kg/cm²) for 24 to 48 hours.

Specimens were then taken out of the containers and split along their diameters. The areas of the portions penetrated by water were colored immediately and photos of the sections were taken. The diffusion coefficient of concrete was obtained by using the following equation:

$$\beta^2 = \frac{D_m^2}{4t\xi^2} \quad (2.1)$$

where β^2 = diffusion coefficient of concrete

D_m = average depth of penetration

t = time elapsed

ξ^2 = constant of integration.

Powers, Copeland, Hayes and Mann [14] have studied the permeability of Portland cement paste. Water was forced into specimens by a constant hydrostatic pressure of about 3 atmospheres and kept under observation until a steady-state flow was reached.

The various apparatuses used by these researchers to measure permeability of concrete were similar in that all of them measured the flow rate of water under pressure through the concrete specimen after a steady flow condition had been reached, with the exception of the method by Murata [13], in which the areas of the portions penetrated by water were used for determining the diffusion coefficient.

In determining the flow rate of water, the inflow method or the outflow method may be used. The rate of inflow can be obtained from the reading on a water column. In the outflow method, the apparatus is arranged to collect the effluent from the free surface and to prevent evaporation losses. The outflow will be less than the inflow in the

early part of any test, because of absorption, but eventually they will agree [10].

The coefficient of permeability of a test specimen can be determined from Darcy's Law [3,4,5,8,10].

$$K = \rho \frac{HQ}{PA} \quad (2.2)$$

where K = coefficient of permeability

ρ = density of water

H = length of test specimen

P = water pressure

Q = rate of flow

A = cross-sectional area of test specimen.

More recently, there is a standard test method, which can be used to rapidly assess the permeability of concrete to chloride ions. This test has been adopted by AASHTO (American Association of State Highway and Transportation Officials) designation: T 277-83 (Standard Method of Test for Rapid Determination of the Chloride Permeability of Concrete) [15]. In this test method, concrete specimens 3.75 inches in diameter and 2 inches in length, which can be cut from cored samples or cast cylinders, is used. The sidewall of the specimen is coated with epoxy. The specimen is then conditioned by vacuum saturation. The ends of the specimen are sealed into an applied voltage cell. A 3% sodium chloride solution is filled into one side of the applied voltage cell and the other side of the cell is filled with a 0.3 N sodium hydroxide solution. A potential difference of 60 volts DC is applied across the specimen for 6 hours. The total charge passed, in coulombs, is obtained

by integrating the area underneath the curve (current in amperes vs. time in seconds) during the 6-hour test period. This total charge is used as an indicator of the chloride permeability of concrete. High chloride permeability is defined as more than 4000 coulombs of charge passed, and low chloride permeability is defined as less than 2000 coulombs of charge passed. An in-depth explanation of this test method can be found in References 15 and 16.

2.2.3 Factors Affecting the Permeability of Concrete

2.2.3.1 W/C-ratio. The permeability of concrete is dependent mainly on the capillary pores in the cement paste. Concretes with lower w/c-ratio result in lower volume of the capillary system than those with higher w/c-ratio [5]. Therefore, the w/c-ratio is one of the major factors affecting the permeability of concrete.

McMillan and Lyse [6] have found the important role played by the quantity of mixing water in controlling the watertightness. An increase in the quantity of mixing water was found to be accompanied by a reduction in watertightness, regardless of the curing period or the characteristics of the cement used.

Ruettgiers et al. [8], Figg [11], Norton and Pletta [7] have also found that the permeability of concrete would increase as the w/c-ratio increased.

Power et al. [14] have studied the permeability of Portland Cement Paste and found that the higher the water/cement-ratio, the higher the permeability. The permeability coefficient of hardened paste ranges from 0.1×10^{-12} to about 120×10^{-12} cm/sec for water/cement-ratios ranging from 0.3 to 0.7.

Goto and Roy [17] have also found that the permeability of hardened cement paste decreased exponentially with decreasing w/c-ratio at a constant curing temperature.

2.2.3.2 Cement type. The type of cement used affects the permeability in so far as it influences the rate of hydration: the more rapid-hardening the cement, the earlier the age at which the permeability is reduced to some desirable value [4]. The permeability of concrete is affected also by the properties of the cement used. For the same water cement ratio, coarser cement leads to a higher porosity than the finer cement. However, the long-term porosity and permeability are unaffected by the type of cement used [4,14]. This is because the degree of hydration would be approximately equal.

McMillan and Lyse [6] found that the development of watertightness at the early ages was comparable to the development of compressive strength. Those cements which gained their strength rapidly during the first few days were found also to gain watertightness more rapidly.

2.2.3.3 Aggregate type. The aggregate used in concrete has a substantial impact on the permeability of concrete. Neville [5] stated that the difference between the permeability of cement paste and of concrete containing a paste of the same w/c-ratio is appreciable as the permeability of the aggregate itself affects the behavior of the concrete. If the aggregate has a very low permeability, its presence reduces the effective area over which flow can take place. Furthermore, since the flow path has to circumvent the aggregate particles, the effective path becomes considerably longer so that the effect of aggregate in reducing the permeability may be considerable. Generally, however, the influence of the aggregate content in the mix is small and

since the aggregate particles are enveloped by the cement paste, in fully compacted concrete, it is the permeability of the paste that has the greatest effect on the permeability of the concrete.

Nyame [18] concluded that the aggregate in concrete can have two opposing effects on permeability. The aggregate particles can obstruct the flow path resulting in reduced permeability, while cracks at the interface between hydrated cement and aggregate and also the porosity of the aggregates can increase the permeability of the concrete.

2.2.3.4 Maximum aggregate size. The maximum aggregate size also influences the permeability of concrete. The larger the maximum aggregate size for a given water/cement-ratio, the higher is the permeability of concrete, probably because of the relatively large water voids developed on the underside of the coarser aggregate particles [10].

Ruettgers, Vidal, and Wing [8] found that for a concrete with a given w/c-ratio and with a maximum aggregate size from $1/4$ to 9 inches, the average relative range in permeability was from about 1 to 30.

Murata [13] also found that the diffusion coefficient of mass concrete with coarse aggregate with a maximum size of 4 inches was on the average about 7 times and 16 times those of concrete with maximum aggregate size of 1.6 inches and 1 inch, respectively.

Gradation of aggregate also influences the permeability of concrete. A well-graded aggregate is more important from the standpoint of watertightness than it is from the standpoint of strength, since a well-graded aggregate will produce a denser concrete [10].

2.2.3.5 Curing condition. The permeability of concrete is decreased when the amount of cement hydrated increased. Continued hydration of the cement results in gel development which reduces the

size of the voids. Therefore, a continued moist curing will decrease the permeability of concrete [10].

Powers et al. [14] found that the chemical reaction between the constituents of Portland cement and water progressively replaces the original cement minerals with hydration products, principally cement gel. The volume of the cement gel produced by hydrating the cement is approximately 2.3 times the volume of the cement. The gel not only replaces the original cement minerals but also tends to fill the original water-filled space. Within a week the coefficient of permeability of cement paste (w/c-ratio = 0.7) dropped to about one one-hundred-thousandth of its initial value. By the 24th day, it had dropped to less than a millionth of its initial value. They also found that the specimens which have been allowed to dry after removal from moist-curing condition and the resaturation of dry specimens will result in increasing the permeability as compared with those which are not subjected to drying and resaturating before being tested. This is because the stresses arising from differential shrinkage caused small cracks on the surface of the specimens.

Ruettggers, Vidal, and Wing [8] found that increasing the age of the test specimen, without interruption in curing to the time when the permeability coefficient was determined, caused a relative reduction in permeability in the ratio of 3 to 1 between the ages of 20 and 60 days and in the ratio of 2 to 1 between the ages of 60 and 180 days, respectively.

McMillan and Lyse [6] have tested mortar specimens cured for 3 days in a moist room and followed by 4, 25 and 177 days in air, giving ages at time of test of 7, 28 and 180 days, respectively. The permeability

of concrete was found to be the same for the same water/cement-ratio. In other words, the 180-day specimens cured only for 3 days in the moist room gave no higher resistance to the penetration of water than the 7-day specimens also cured for the first 3 days in the moist room. This result is not in agreement with the result by Powers et al. [14].

Nagataki and Ujike [19] have studied the air permeability of concrete specimens with 28-day and 90-day water curing and followed by 21, 84 and 119 days air curing. They found that the 28-day water-cured specimens produced higher air permeability than those 90-day water cured, and the longer periods of drying specimens produced higher air permeability than those shorter periods of drying when compared at the same water-cured period.

Permeability of steam-cured concrete is generally higher than that of moist-cured concrete, except for concrete subjected to a long curing temperature cycle [3].

Higginson [20] found that the permeability of concrete specimens subjected to 24 hours of steam curing at 160°F and then 28 days of curing in air were greater than that of the companion specimens continuously fog cured for 28 days.

2.2.3.6 Silica fume. Silica fume is a by-product resulting from the reduction of high-purity quartz with coal in electric arc furnaces in the production of silicon and ferrosilicon alloys. The fume, which has a high content of amorphous silicon dioxide and consists of very fine spherical particles, is collected from the gases escaping from the furnaces. Silica fume is also collected as a by-product in the production of other silicon alloys such as ferrochromium, ferromanganese, ferromagnesium, and calcium silicon [21].

Silica fume varies from light to dark gray in color. The specific gravity of a typical silica fume is about 2.2. The bulk density of silica fume ranges from 15.6 to 18.7 lb/ft³. The fineness of silica fume is of the order of 20,000 m²/kg when measured by nitrogen adsorption techniques. Because of its extreme fineness and high silica content, silica fume is a highly effective pozzolanic material [21].

In general applications, part of the cement may be replaced by a much smaller quantity of silica fume without loss of strength. As a result of silica fume addition, there may be an increase in the water demand. If it is desired to maintain the same water-cementitious material ratio, water-reducing or high-range water-reducing admixtures should be used to maintain the required slump with no change in the water-cementitious material ratio. Silica fume has been successfully used to produce very high strength, low permeability, and chemical resistance in concrete [21]. Mehta and Gjorv [22] have shown that mortars and concretes incorporating silica fume are less permeable. It has been shown that this is due to a decrease in the number of coarse pores of the cement-silica fume paste system although the total porosity remains nearly the same as in neat cement paste. This fact was also found by Yamato et al. [23].

Carlsson et al. [24] stated that concrete with silica fume can improve the resistance to freezing-thawing attack, chloride penetration and deicer scaling. This is because the use of silica fume results in lower permeability and free lime content in the cement paste.

According to Sellevold [25], the reasons that the use of silica fume in concrete leads to reduced permeability are (1) silica fume produces a calcium silicate hydrate gel with a lower calcium-to-silica

ratio than the cement hydration, and (2) silica fume has a definite filler effect that is believed to distribute the hydration products more uniformly in the available space. The silica fume in concrete results in decreased permeability more than it results in increased compressive strength, particularly for low content levels of silica fume and of low concrete strength levels.

Hooton [26] found that for three pozzolans, namely fly ash, silica fume and slag, the silica fume not only was more effective in reducing permeability at early age, but also was the most effective in reducing calcium hydroxide contents of the cement pastes. The decrease in the amount of calcium hydroxide in cement paste contributes to increased sulfate and chloride resistance.

Radjy et al. [27] suggested that the use of silica fume to replace cement should be less than 10% by weight of cement. The use of higher than 10% of silica fume in concrete will result in sticky mixes which are difficult to work with, high susceptibility to skin formation and plastic shrinkage during setting, and excessive drying and chemical shrinkage. The Norwegian standards limit the silica fume to a maximum of 10% by weight of cement.

Ozyildirim [28] has shown that concrete containing silica fume at a low replacement rate of 5% and at a water-cementitious material ratio of 0.4 or less can provide satisfactory strength and low permeability for bridge deck overlays with a minimum thickness of 1 1/4 inch.

Nagataki and Ujike [19] have studied the air permeability of concretes mixed with silica fume. They found that the coefficients of air permeability of concretes with 10%, 20% and 30% replacement of silica fume are lower than the control concrete for 28-day and 91-day

water-cured specimens. The observed effect of silica fume on air permeability is similar to that on the water permeability of concrete.

2.2.3.7 Fly ash. Fly ash is the ash precipitated electrostatically from the exhaust fumes of coal-fired power stations, and is the most common artificial pozzolana [5].

ASTM C618 [29] divides fly ashes into two classes, namely Class F and Class C. The chemical composition and physical properties are used to classify the fly ashes. Generally, Class F consists of less than 10% of CaO while Class C consists of more than 10% of CaO [30].

When fly ash concrete is properly cured, fly ash will react with available calcium and alkali hydroxides in cement paste to produce calcium silicate hydrate. This results in reducing the size of the pore spaces in the cement paste in concrete and also reducing the risk of leaching of calcium hydroxide. As a result, the permeability and the rate of diffusion of moisture and aggressive chemicals into the concrete is reduced, thereby reducing the danger of damage due to sulfate attack, steel corrosion and alkali-silica reaction [31].

Hooton [26] has studied permeability of cement pastes containing fly ash, slag and silica fume. He found that the permeability of paste with 35% fly ash is significantly lower than that of paste made with sulfate-resisting portland cement and paste with 65% slag and slightly higher than that of paste with 10% silica fume.

According to the literature review by Gebler and Klieger [32], concrete age is a major factor in the watertightness of fly ash concrete. This is because the pozzolanic reaction of fly ash produces calcium silicate hydrates that fill the pore spaces in cement paste, resulting in improved watertightness.

Maslehuddin et al. [33] have studied the effects of fly ash addition on the corrosion-resisting characteristics of concrete and found that the fly ash addition to concrete mixes improves the long-term corrosion resistance of concrete but at the early age showed higher corrosion activity as compared to plain concrete. This could be attributed to the higher permeability of fly ash concrete, as compared to the plain concrete at initial ages due to reduction in the quantity of cement. The reaction of fly ash with calcium hydroxide, which produces a dense concrete and thus inhibits the ingress of chloride ions, takes place at a slower rate.

Marsh et al. [34] have investigated the permeability of cement paste containing fly ash at different ages of test and temperature of water curing. The results showed that pastes with fly ash produced substantially lower permeability than those without fly ash at 7-day and longer periods of water curing and temperature of 120 and 150°F. For water curing at a temperature of 70°F, pastes with fly ash produced higher permeability at 7 and 28 days but significantly lower permeability at 91-day and longer periods of water curing than those without fly ash.

The results of tests by Nagataki and Ujike [19] showed that the coefficient of air permeability of fly ash concrete is about the same as that of the control concrete at 28-day water curing. At 90-day water curing, the fly ash concrete is more airtight than the control concrete.

2.3 Modulus of Rupture of Concrete

2.3.1 Test Methods

There are two standard ASTM test methods for determining the modulus of rupture or flexural strength,

1. ASTM C78--flexural strength of concrete (using simple beam with third-point loading), and
2. ASTM C293--flexural strength of concrete (using simple beam with center-point loading).

Using the linear flexure formula, the maximum tensile stress reached in the bottom fiber of a test beam is then computed. This stress is referred to as the modulus of rupture [4]. It is expressed as

$$f_r = \frac{PL}{bd^2} \quad \text{for ASTM C78} \quad (2.3)$$

$$\text{or } f_r = \frac{3}{2} \frac{PL}{bd^2} \quad \text{for ASTM C293} \quad (2.4)$$

where f_r = modulus of rupture

P = maximum applied load

L = span length

b = average width of specimen

d = average depth of specimen.

Wright [35] found that the modulus of rupture under central loading was 20% to 25% higher than that under third point loading. The third point loading was found to produce more uniform test results, the

coefficient of variation of the modulus of rupture for the central loading being approximately one-third higher.

2.3.2 Relationship Between Compressive Strength and Modulus of Rupture

The compressive strength and modulus of rupture are closely related, but there is no direct proportionality. As the compressive strength of concrete increases, the modulus of rupture also increases but at a decreasing rate. In other words, the ratio of modulus of rupture to compressive strength depends on the general level of the compressive strength; the higher the compressive strength, the lower the ratio. The range of the ratio of modulus of rupture to compressive strength (f_r/f'_c) is approximately 0.11 to 0.23 when the modulus of rupture is determined under third-point loading [4,5].

Several factors influence the relation between the two strengths. The ratio of modulus of rupture to compressive strength is greater for concretes made with crushed coarse aggregate. The properties of the fine aggregate and the overall grading of the aggregate also influence the ratio of modulus of rupture to compressive strength. Age is also a factor in the relation between modulus of rupture and compressive strength. Beyond about one month, the modulus of rupture increases more slowly than the compressive strength, so that the ratio of modulus of rupture to compressive strength decreases with age. The modulus of rupture is more sensitive to inadequate curing than the compressive strength, possibly because the effects of nonuniform shrinkage of flexural test beams are very serious. Air-cured concrete has a lower ratio of modulus of rupture to compressive strength than concrete cured in water and tested wet [4,5].

A number of empirical formulas relating the compressive strength (f'_c) to the modulus of rupture (f_r) have been suggested. Many of them are of the form

$$f_r = k(f'_c)^n \quad (2.5)$$

The European Concrete Committee (CEB) has recommended values of $k = 9.5$ and $n = 0.5$ [4], while the ACI Code [36] has suggested values of $k = 7.5$ and $n = 0.5$.

Another formula was suggested at the University of Illinois [4,5,37] as

$$f_r = 3000 / (4 + 12000/f'_c) \quad (2.6)$$

Jones and Kaplan [38] found that the relation between the modulus of rupture and compressive strength depends on the type of coarse aggregate used since the properties of aggregate, especially its shape and surface texture, affect the ultimate strength in compression very much less than the strength in tension.

2.3.3 Relationship Between Modulus of Rupture and Splitting Tensile Strength

The splitting tensile strength ranges approximately from 50% to 75% of the modulus of rupture with higher-strength concrete having relatively higher ratios of tensile strength to modulus of rupture [10].

Narrow and Ullberg [39] found that there were good and consistent relationships between modulus of rupture and splitting tensile strength of the concrete. The strength relationship was not affected by the type of aggregate used nor by age of the concrete specimen when tested.

However, there was an appreciable reduction in the flexural to tensile strength ratio with increase in splitting tensile strength of the concrete.

2.3.4 Factors Affecting the Modulus of Rupture

Since the modulus of rupture and compressive strength are closely related, the factors that influence the compressive strength also influence the modulus of rupture. Factors known to influence compressive strength include w/c-ratio, cement type, aggregate type, maximum aggregate type, curing condition and admixture. These factors are discussed in more detail at this point.

2.3.4.1 W/C-ratio. The strength of concrete is dependent largely on the capillary porosity or gel-pore ratio, which are not easy quantities to measure or predict. However, the capillary porosity of a properly compacted concrete at any degree of hydration is determined by the w/c-ratio. Therefore, in practice, we can assume the strength of properly compacted concrete at a given age by specifying the w/c-ratio [40].

Duff Abrams found that a relation existed between w/c-ratio and concrete strength. Popularly known as Abram's w/c-ratio rule, this inverse relation is represented by the expression [41],

$$f_c = k_1/k_2^{(w/c)} \quad (2.7)$$

where w/c represents the water/cement-ratio

k_1 and k_2 are empirical constants.

In general, the higher the w/c-ratio, the lower the strength of concrete, or the lower the modulus of rupture.

2.3.4.2 Cement type. The effect of cement type on concrete strength depends on the chemical composition and fineness of cement [40]. Since the concrete strength comes primarily from the hydration of two compounds, C_3S and C_2S , and the rate of hydration increases with increasing fineness of cement, the effect of cement type on strength development is considerable for ages up to one month and becoming less important at later ages.

2.3.4.3 Aggregate type. Although the w/c-ratio is the most important factor affecting strength, the properties of aggregate cannot be ignored. The aggregate parameters that are most important are the shape, texture and the maximum aggregate size. The aggregate texture depends on whether the aggregates are naturally occurring gravels, which tend to be smooth and round, or crushed stones, which tend to be rough and angular [40].

Mehta [41] stated that a concrete mixture containing a rough-textured or crushed aggregate would show somewhat higher strength (especially flexural strength) at early ages than a corresponding concrete containing smooth or naturally weathered aggregate of similar mineralogy. A stronger physical bond between the aggregate and the hydrated cement paste is assumed to be responsible for this. At later ages, when chemical interaction between the aggregate and the cement paste begins to take effect, the effect of the surface texture of aggregate on strength may be reduced.

Jones and Kaplan [38] found that concretes containing smooth gravels began to crack at lower stresses than those containing coarser-textured aggregate. Concretes which best resisted pre-cracking in compression also exhibited the highest modulus of rupture. The

mechanical interlocking of coarse aggregate in concrete resulted in higher compressive strength than the comparable mortar. However, the presence of aggregate in concrete resulted in a lower modulus of rupture than the comparable mortar. The cracks at the interface between the aggregate and cement paste are believed to be the factor that caused the reduced modulus of rupture.

2.3.4.4 Maximum aggregate size. The use of a larger maximum aggregate size affects the strength in several ways. Because the use of larger particles reduces the specific surface area of the aggregate, the bond strength is also less, and this tends to reduce the strength. Also, larger aggregate particles provide more restraint on volume changes in the paste, and thus may induce additional stresses in the paste, which tend to weaken the concrete [40].

Cordon and Gillispie [42] found that, in the range of No. 4 sieve to 3-inch sieve, the effect of increasing maximum aggregate size on the 28-day compressive strength of concrete was more pronounced with a high-strength and a medium-strength concrete than with a low-strength concrete. Mehta [41] stated that in a given concrete mixture, at a constant w/c-ratio, the tensile-compressive strength ratio would increase with the decreasing size of coarse aggregate.

2.3.4.5 Curing condition. The necessity for curing arises from the fact that hydration of cement can take place only in water-filled capillaries. This is why a loss of water by evaporation from the capillaries must be prevented. Furthermore, water lost internally by self-desiccation has to be replaced by water from outside, and ingress of water into the concrete must be made possible [5].

Mehta [41] stated that the longer the moist-curing period, the higher the strength will be, assuming that the hydration of cement particles is still going on.

Nielson [2] found that the modulus of rupture of the beams cured in water for 21 days with additional air curing for 7 days has a tendency to decrease. This tendency of the general decrease in modulus of rupture upon transfer from water curing to air curing may be explained by the fact that, when air curing begins, the top surface layer dries out and tensile stresses are set up in the surface layer which result in reduced modulus of rupture.

The strength of concrete is also affected by the curing temperature. The temperature in the range of 70 to 75°F is the standard range, according to ASTM C192 [43]. The temperatures lower than this range result in lower early strength and higher later strength as compared to the strength at the standard temperature, while the reverse is true for the curing temperatures higher than the standard temperature. The increased early strength at higher temperatures is due to the fact that the chemical reactions of hydration are speeded up. The nonuniform distribution of hydration products within the paste is believed to be the reason why the later strength is decreased for the higher temperatures [5,10,40]. Sellevold [25] stated that curing at high temperatures has a greater accelerating effect on the silica fume concrete than the control concrete. A curing temperature of about 120°F is needed for silica fume concrete to have approximately the same strength as the control concrete. From the extensive study on properties of fly ash concrete by Gebler and Klieger [44], the results show that the

compressive strength of fly ash concrete at 7 days and 1 year increased with an increase in curing temperature.

Higginson [20] found that the compressive strength of specimens which were steam cured at 160°F for 12 hours and followed by air curing for 5 days was approximately equal to that obtained with fog curing for 7 days.

Curing compounds have become very widely used in curing concrete structures. They can be used to retard or reduce the loss of moisture from the concrete. Care must be taken in order to avoid the pinholes and in properly overlapped joints [30,40]. The presence of pinholes and joints can lead to microcracks on the surface of concrete members due to drying shrinkage, and a low degree of hydration due to the lack of water for the hydration process.

2.3.4.6 Silica fume. Silica fume has been successfully used to produce very high-strength concrete because of its extreme fineness and high silica content. The main contribution of silica fume to concrete strength development at normal curing temperatures takes place from about 3 to 28 days. Silica fume concrete gains strength very much like concrete containing fly ash or slag. The important difference is that silica fume appears to be a more efficient pozzolan. The pozzolanic reactions of silica fume are evident at earlier ages than those of fly ash [21].

Malhotra and Carette [45] stated that silica fume concrete using superplasticizer produces about the same compressive strength as the control concrete at 1-day curing and produces higher compressive strength than the control concrete at 28-day curing and beyond. Silica fume is generally more efficient in concretes with high w/c-ratios and

superplasticizer. Cement of 3 to 4 lbs. can be replaced by silica fume of 1 lb. without reducing the compressive strength of the concrete.

Yamato et al. [23] found that the compressive strength of mortar with silica fume was lower than the control mortar without silica fume at curing periods of 7 days. At 28 and 91 days, the mortars with silica fume showed higher compressive strength and modulus of rupture than those of the control mortars. These results are generally in agreement with those of Malhotra and Carette [45], Sandvik and Gjorv [46] and Sellevold [25].

Yogendran et al. [47] have studied the use of silica fume in high-strength concrete. They found that the 15% replacement of cement by silica fume produced the highest compressive strength at all ages, when a superplasticizer was used to adjust slump to a target slump. At a percent replacement of 20 and higher, the compressive strengths of silica fume concrete were lower than that of the control concrete.

The strength-development pattern of modulus of rupture and splitting tensile strength of silica fume concrete is similar to that for compressive strength [21,25,45]. However, if a silica fume concrete is exposed to drying after one day of curing, the modulus of rupture and splitting tensile strength are reduced more than those of the control concrete [25].

Ozyildirim [28] found that concrete containing silica fume at a low replacement rate of 5% and at a w/c-ratio of 0.4 can increase the modulus of rupture by about 7% of that of the control concrete.

Mehta and Gjorv [22] stated that silica fume which is a highly reactive pozzolan, can be used to improve early strengths and durability of concrete containing fly ash or slag. The results from their study

showed that concrete containing silica fume and fly ash produced about the same compressive strength as the control concrete at 7-day moist curing, and produced higher compressive strength than the control concrete by about 40% at 28-day moist curing. The compressive strength of fly ash concrete was lower than the control concrete by about 10%, while the silica fume concrete showed higher compressive strength than the control concrete by about 75% at 28-day moist curing.

2.3.4.7 Fly ash. The fly ash in concrete reacts with calcium hydroxide very slowly at early ages. This results in lower compressive strength at early ages. At later ages, the fly ash reacts with calcium hydroxide to produce calcium silicate hydrate, which produces denser cement paste, and leads to higher compressive strength [5,31,44].

Tikal'sky et al. [48] have studied strength and durability of fly ash concrete. Cement was replaced by an equal weight of fly ash at the levels of 0% to 35%. They found that the modulus of rupture of fly ash concrete is slightly lower than the control concrete at early ages.

Naik and Ramme [49] have investigated the high-strength concrete containing quantities of fly ash up to 60% of total cementitious content. Class C fly ash was used to replace cement in the ratio of 1.25:1 (fly ash:cement) by weight. The results showed that the fly ash concretes at all levels of replacement generally produced higher compressive strength than the control concrete at 28 days and beyond. The compressive strength of fly ash concrete with 40% replacement was the highest for most of the cases. They suggested that the fly ash concrete with a high percentage of fly ash should not be used for structures where forms need to be removed at the early ages.

CHAPTER 3 DEVELOPMENT OF THE PERMEABILITY TEST SET-UP

This chapter describes the development of the laboratory permeability test set-up which was used in the laboratory testing program of this research project. The permeability test procedure and the method of calculation to obtain the coefficient of permeability from the raw data are also presented here.

3.1 Development of a Suitable Permeability Apparatus

3.1.1 Basic Requirements

In the development of a suitable permeability apparatus, the following requirements and concerns were considered:

1. The apparatus should be easily duplicated.
2. The concrete specimens could be inserted and removed with a minimal loss of time.
3. Concrete specimens cut from larger-size concrete specimens could be used.
4. The small quantity of water flowing through the concrete specimen could be measured accurately.

Various different permeability apparatuses were designed, built and tested, based on the considerations listed above. The following sections describe the various designs which have been used in experiments, as well as the final permeability apparatus developed.

3.1.2 First Design Tested

The first permeability apparatus built is shown in Figure 3.1. Cylindrical concrete specimens 4 inches in diameter and 2 inches in height were cast. The sides of the test specimens were wire-brushed until all loose particles have been removed. Care was taken to ensure that the surfaces of the test specimens were dry. The cylindrical wall of the specimen was sealed with an epoxy casting about $\frac{3}{8}$ -inch thick by means of an aluminum casting mold. The concrete specimen with the epoxy casting was then inserted into an aluminum ring which was then bolted to the base plate. The aluminum ring had three O-ring seals--one was placed between the ring and the base plate and the others were placed between the ring and the epoxy resin. The ring-epoxy-concrete assembly was contained in a steel pipe covered with a flat steel plate. The steel pipe was provided with two O-ring seals--one was between the pipe and the cover plate and the other was between the pipe and the base plate. The cover plate was bolted to the base plate by half-inch bolts. The apparatus was supported by a support stand. In the expected operation of the test, pressurized water in the steel pipe would flow vertically downward through the concrete, and the coefficient of permeability could then be calculated from the rate of flow through the specimen. When the apparatus was tested with compressed air up to 60 psi, leakage through the aluminum ring bolt holes was noted. Subsequently, silicone was used to seal the bottom part of the aluminum ring and O-rings were used to seal the bolt holes. However, these measures failed to stop the leakage completely. Though other modifications could still be tried to stop the leakage, it was decided that it was not the most effective set-up and other improved designs were tried out.

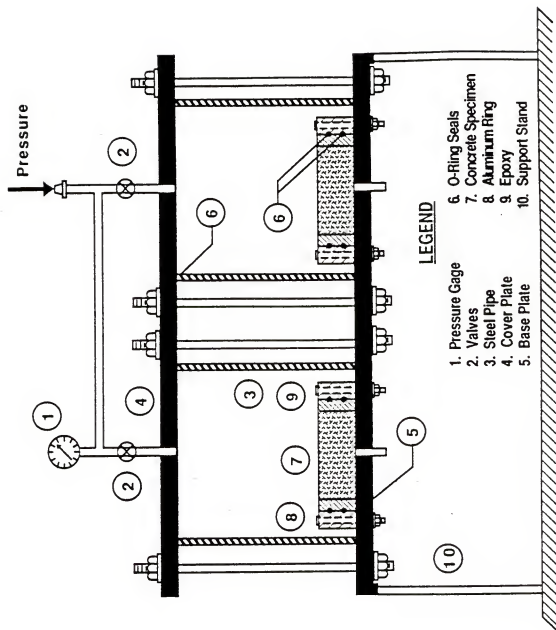


Figure 3.1 Schematic of the First Permeability Test Apparatus

3.1.3 Second Design Tested

The second permeability apparatus built is shown in Figure 3.2. It consisted of a PVC pipe 4 inches in diameter and 4 inches in height and a PVC cap. The bottom of the PVC pipe was closed by a PVC flat plate. A cylindrical concrete specimen, 3 inches in diameter and 2 inches in height, was used. The surface of the specimen was wire-brushed until all loose particles had been removed and kept dry. The concrete specimen was placed in the center of the PVC pipe and the gap was filled with epoxy. When the epoxy had hardened, the PVC cap was connected to the PVC pipe. No leakage was found when a pressure of up to 50 psi was applied. However, when an air pressure of 60 psi was applied, a crack was formed between the inside surface of the PVC pipe and the epoxy. This was due to excessive expansion of the PVC pipe.

3.1.4 Third Design Tested

The third permeability apparatus built is shown in Figure 3.3. It is similar to the second design with the exception that a metal pipe with internal threads was used in lieu of the PVC pipe. A casting mold made with the same metal pipe was used to cast the threaded epoxy around the concrete specimen. The specimen with the threaded epoxy casting was sealed with Teflon® tape around the threads and turned into the metal pipe. The cover plate was then connected to the metal pipe. This system worked with no leakage up to 60 psi. With a pressure above 60 psi, water was found leaking through the threads.

3.1.5 Final Design Adopted

Through further experimentation, a permeability apparatus of satisfactory performance was finally developed. The final permeability apparatus used is shown in Figure 3.4. It consists of a 1-inch thick

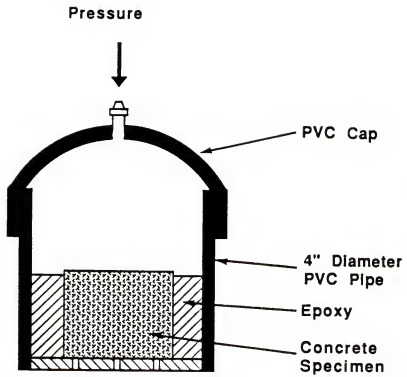


Figure 3.2 Schematic of the Second Permeability Test Apparatus

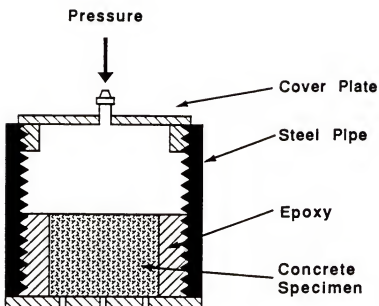


Figure 3.3 Schematic of the Third Permeability Test Apparatus

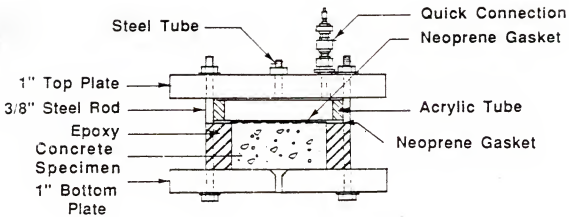
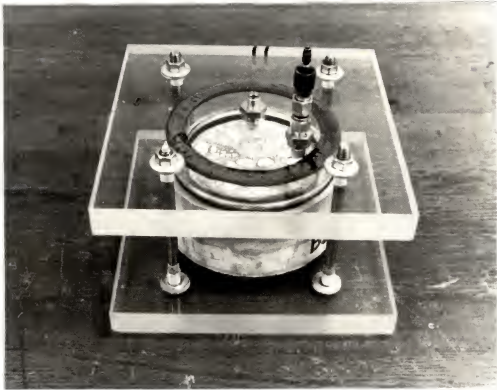


Figure 3.4 Final Permeability Test Apparatus

acrylic top plate, a 1-inch thick acrylic base plate, an acrylic tube (with an inside diameter of 4 1/2 inches, a thickness of 1/2 inch and a height of 1 inch), four steel bolts (6 1/2 inches long and 3/8 inch in diameter) and four matching nuts. The top and bottom of the acrylic tube are provided with neoprene gaskets. One gasket is placed between the bottom of the acrylic tube and the top of the specimen and the other one is placed between the top of the acrylic tube and the acrylic top plate. A steel connection tube is inserted through the center of the acrylic top plate to provide a connection between the water in the acrylic tube to the pressure source and the manometer tube which measures the amount of water going into the specimen. A quick connection is installed on the top acrylic at 2 inches from the center. It is used for refilling water into the manometer tube.

Twenty permeability apparatuses as described above were built and connected to twenty manometer tubes which were held in a vertical direction against two boards. All the manometer tubes were connected to a main pressure line which runs along the top of the vertical boards. The pressure is held constant by a regulator and is indicated on a pressure gage which is attached to one side of the vertical board for ease of reading. Any line can be disconnected from the main pressure line at any time without interfering with the operation of the other lines by turning off the individual valves. A picture of the test set-up is shown in Figure 3.5.

Another regulator and pressure gage are used to regulate the pressure in the water reservoir. The water reservoir, which is used for refilling water into the manometer tube, consisted of an acrylic tube 8 inches in diameter and 14 inches in length, top and bottom steel plates,

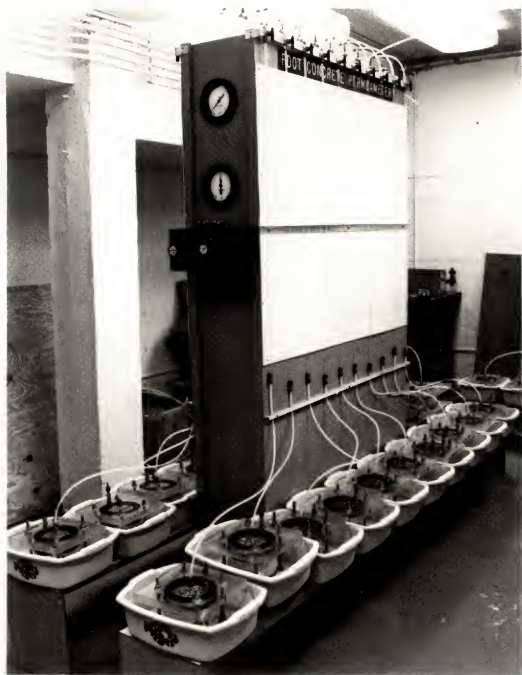


Figure 3.5 Permeability Test Set-Up

two quick connections (one on each end), eight $\frac{3}{8}$ -inch x 18-inch threaded rods and eight matching nuts. The water reservoir used is shown in Figure 3.6.

3.2 Permeability Test Procedure

3.2.1 Preparation of Test Specimens

The test specimens are cylindrical specimens of 4 inches in diameter and 2 inches in height, and are made by cutting 8-inch cast cylinders at 1 and 3 inches from the bottom surface. The circumference of the test specimens are roughened and wire-brushed until all loose particles have been removed. Care is taken to ensure the surfaces of the test specimens are dry.

A 1-inch thick layer of epoxy (Sikadur 32, Hi-Mod) is to be coated around the side of the test specimen by means of a casting mold. The casting mold is 6 inches in diameter and 2 $\frac{1}{4}$ inches in height and is bolted down to a steel plate. The inner surface of the casting mold and the top surface of the steel plate are coated with a very thin layer of release agent. The side of the specimen is first coated with a thin layer of epoxy. The specimen is then placed in the center of the casting mold, and the remaining gap between the casting mold and the test specimen is filled with epoxy. The air bubbles are eliminated from the epoxy by using a small hand vibrator. When the epoxy has hardened, the test specimen is taken out of the casting mold. The small amount of epoxy which may adhere to the top and bottom surfaces of the test specimen is to be removed. Figure 3.7 shows a casting mold used. A picture of a prepared concrete specimen coated with epoxy on the side is shown in Figure 3.8.

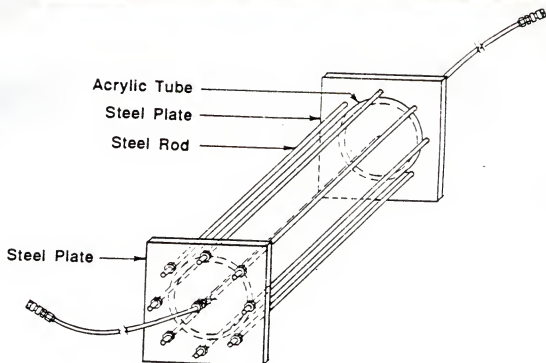
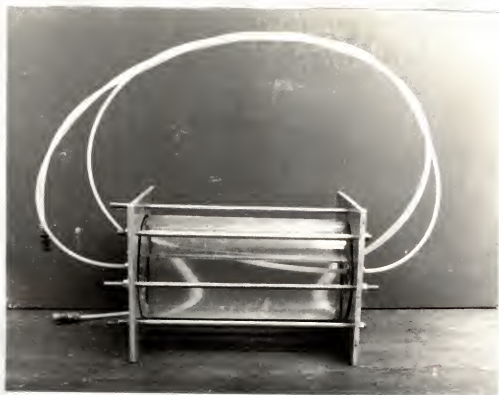


Figure 3.6 Water Reservoir

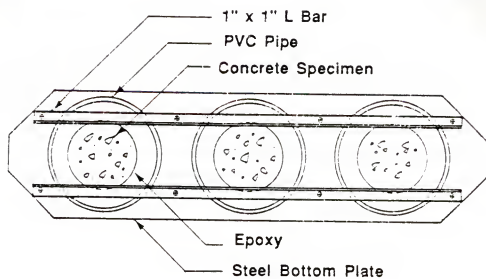


Figure 3.7 Casting Molds



Figure 3.8 Prepared Concrete Specimens

3.2.2 Testing Procedure

The prepared test specimen is placed into the permeability apparatus in the following steps:

1. The test specimen is placed on the center of the base plate.
2. A neoprene gasket is placed on the specimen.
3. The acrylic tube is placed on the neoprene gasket.
4. Another neoprene gasket is placed on the acrylic tube.
5. The top plate is placed on the neoprene gasket and bolted to the base plate with four $\frac{3}{8}$ -inch diameter rods (see Figure 3.4).
6. A torque of approximately 10 to 12 foot-pounds is applied to all bolts, and this needs to be checked periodically in order to ensure the same torque during the entire period of testing.

The permeability apparatus is then connected to the manometer tube in the following procedure:

1. The valve for the manometer tube is turned off.
2. The manometer tube is connected to the connection at the center of the top plate of the permeability apparatus.
3. Pressure is applied to the main pressure line and maintained at 100 psi by the regulating valve, which controls the pressure from the air compressor.
4. The water reservoir is connected to the other pressure regulator through one of the two quick connections of the water reservoir and a pressure of 102 psi is maintained.
5. The other quick connection of the water reservoir is connected to the quick connection at the top plate of the permeability apparatus, and water is injected into the permeability

apparatus and the manometer tube by opening the manometer valve slowly until the water in manometer tube reaches the top level. The valve is then turned off, and the permeability apparatus is disconnected from the water reservoir.

6. The air bubbles which are trapped inside the permeability apparatus are to be released through the quick connection at the top plate.

At this stage, the permeability test can be started by simply opening the manometer valve to connect the 100 psi air pressure line to the manometer. The water level in the manometer tube and the time at the start of the test are recorded. Possible leakage of water between the epoxy and the side of the test specimen is to be checked. If leakage occurs, the specimen has to be discarded. However, occurrence of leakage has been very infrequent. The test is continued if no leakage is found. When the water level in the manometer tube reaches a low level, additional water can be added to the manometer tube from the water reservoir. The process of refilling the manometer tube is the same as the process of initial filling as described earlier.

3.2.3 Method of Measuring Rate of Flow

The amount of water flowing into a test specimen is measured by reading the water level of the $\frac{1}{8}$ -inch manometer tube at least once a day. A plot of the cumulative amount of water flowing into the test specimen vs. time is drawn for each test specimen, to determine when a steady state flow condition will have been reached. The steady state condition is usually reached within a time of about 10 to 21 days which is dependent upon many factors such as the w/c-ratio and curing condition. The test is continued for about 7 to 10 days beyond the steady

flow condition. The average rate of inflow in the last 7 to 10 days is used as the rate of inflow for the test specimen. Two typical plots of cumulative amount of water flow vs. time are shown in Figure 3.9.

The evaporation of water introduces complications to the measurement of water inflow in the following two ways:

1. The water in the manometer may evaporate into the compressed air line, resulting in a drop in the manometer water level.
2. After water has flowed through the test specimen, the water at the bottom surface of the specimen may evaporate and produce a slight suction which can increase the rate of flow.

The amount of water evaporating into the air line is estimated by using a reference permeameter which is connected to a manometer tube of the same size and the same main pressure line. The reference permeameter contains no test specimen and has a base plate with no hole, creating a condition of zero flow. The reference permeameter is filled up with water and connected to a manometer tube and the main pressure line in a similar manner as for the other permeameters. The water level in the reference manometer tube is read periodically to estimate the amount of water evaporating into the air line, which is then used to adjust the measured rate of inflow for the other permeameters.

The problem of the effects of evaporation of water from the bottom surface of the test specimen is dealt with by soaking the lower portion of the permeameter in water at about 4 to 5 days after the test is started. The reason the permeameter is not soaked in water immediately after the start of test is that leakage of the permeameter has to be checked for in the first few days.

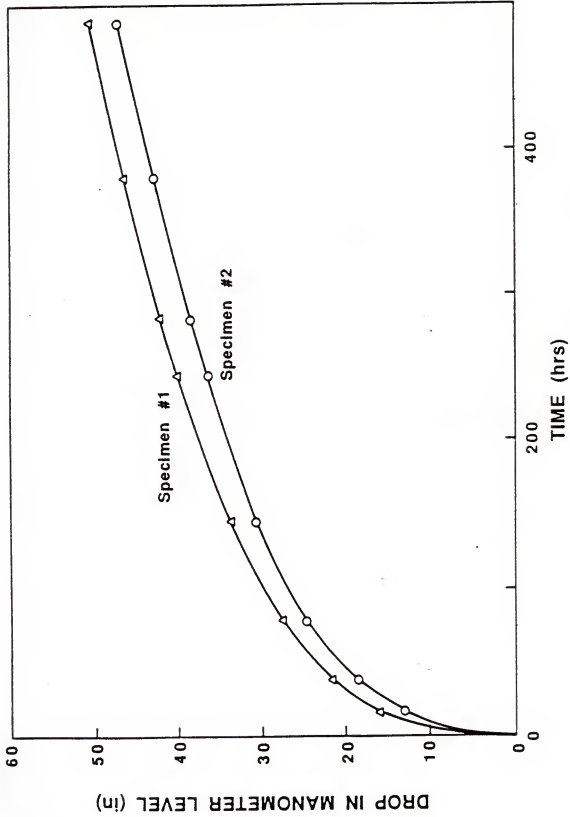


Figure 3.9 Typical Plots of Manometer Readings in Permeability Tests

3.2.4 Calculation of Coefficient of Permeability

The coefficient of permeability of a test specimen is computed from the net rate of inflow by using the following equation which is based on Darcy's Law:

$$K = \rho \frac{H}{P} \frac{Q}{A} \quad (3.1)$$

where K = coefficient of permeability in in/sec or cm/sec.

ρ = density of water in lb/in³ or kg/cm³.

H = length of test specimen in inches or cm.

P = water pressure in psi or kg/cm².

Q = net rate of inflow in in³/sec or cm³/sec.

A = cross-sectional area of test specimen in in² or cm².

3.3 Effects of Specimen Height on Permeability Test Results

During the course of development of the laboratory permeability apparatus, a question arose as to what the effect of specimen height was on the results of the permeability test. An experimental study was thus conducted in order to answer this question. A two-way full factorial experimental design was used. The two factors which were incorporated in the factorial design were (1) specimen height and (2) maximum aggregate size. Three specimen heights, namely 1.5, 2.0 and 3.0 inches, and three maximum aggregate sizes, namely $\frac{3}{8}$, $\frac{3}{4}$ and 1 inch, were used. Two replicate samples per combination of specimen height and maximum aggregate size were tested in the permeability apparatus. Thus, a total of 18 specimens were tested in this study. The following conditions were fixed:

1. Aggregate type: Brooksville
2. Concrete class: Florida Class IV
3. Cement content: 752 lb/yd³
4. Water/Cement-ratio: 0.33
5. Cement type: Type I
6. Curing condition: 16-hour steam curing at 160°F

The concrete specimens were evaluated for their coefficients of permeability in accordance with the procedure as described in Section 3.2. The results of this randomized factorial experiment are displayed in Table 3.1. From the data shown in Table 3.1, it can be noted that the mean coefficient of permeability of the concrete with one maximum aggregate size is quite different from that of a concrete with another maximum aggregate size. However, the variation of the data due to the use of different specimen heights, as measured by the standard deviation, is relatively smaller than the difference due to the effect of different aggregate sizes. In other words, the effect of maximum aggregate size is much greater than the effect of specimen height on the permeability test results.

The following linear model is used for statistical analysis of the permeability data:

$$Y_{ijk} = \mu + A_i + H_j + AH_{ij} + \epsilon_{ijk} \quad (3.2)$$

where Y_{ijk} = permeability coefficient of concrete of i^{th} aggregate size, j^{th} specimen height and k^{th} replicate

μ = the overall mean effect

A_i = the main effect of the i^{th} maximum aggregate size

H_j = the main effect of the j^{th} specimen height

Table 3.1 Coefficients of Permeability of Concrete Specimens

Maximum Aggregate Size (inches)	Specimen Height (inches)			Means ($\times 10^{-12}$ in/sec)	Standard Deviation ($\times 10^{-12}$ in/sec)
	1.5	2.0	3.0		
$\frac{3}{8}$ ($\times 10^{-12}$ in/sec)	5.13	5.51	5.83	5.39	0.614
	5.18	4.47	6.24		
$\frac{3}{4}$	6.37	6.55	7.38	7.21	0.654
	8.04	7.20	7.74		
1	10.48	14.60	11.26	12.54	2.10
	10.21	14.56	14.11		

Note: Concrete Class: Florida Class IV, w/c-ratio = 0.33
Aggregate Type: Brooksville
Cement Type: Type I
Curing Condition: 16-hour steam curing at 160°F

AH_{ij} = the effect of interaction of the i^{th} aggregate size
with the j^{th} specimen height

ϵ_{ijk} = random error

i, j = 1, 2, 3

k = 1, 2

Analysis of Variance (ANOVA) was performed on the test data using the linear model above. The most important results of an ANOVA are the p-values for the factors considered. Basically, the p-value indicates the probability of error of the statement that the factor has a significant effect on the measured parameter. For example, if a factor A has a p-value of 0.01, the statement that factor A is significant will have a probability of error of 0.01. A lower p-value indicates a higher level of significance. Normally, a probability of error of 0.05 is used. If the p-value of a factor is equal to or less than 0.05, the factor is considered to be significant.

The results of ANOVA are summarized in Table 3.2. From Table 3.2, it is seen that the effects of maximum aggregate size and the interaction of maximum aggregate size by specimen height (AB) are significant while the effect of specimen height is not significant. The ANOVA results indicated that while the effect of specimen height is generally not significant, it may be significant for certain aggregate sizes.

To examine this problem further a comparison of means for specimen height at each maximum aggregate size was performed using Duncan's test at a level of significance of 0.05. The results are summarized in Table 3.3. From Table 3.3, it is seen that the permeability of three specimen heights are not significantly different from each other for the maximum aggregate sizes of $3/8$ inch and $3/4$ inch. For maximum aggregate size of

Table 3.2 Results of ANOVA on the Permeability Test Data

Source	df	MS ($\times 10^{-24}$)	F-Value	P-Value
Aggregate Size, A	2	86.68	116.35	0.0001*
Specimen Height, B	2	2.98	4.19	0.0517
Interaction, AB	4	3.44	4.85	0.0232*
Error, ϵ	9	0.711		

Note: * Significant at α level of 0.05

Table 3.3 Grouping of Specimen Height by Maximum Aggregate Size on the Basis of Coefficient of Permeability from Duncan's Test

Maximum Aggregate (inches)	Specimen Height (inches)	Means ($\times 10^{-12}$ in/sec)	Grouping
$3/8$	1 $1/2$	5.155	A
	2	4.99	A
	3	6.035	A
$3/4$	1 $1/2$	7.205	A
	2	6.875	A
	3	7.56	A
1	1 $1/2$	10.345	B
	2	14.58	A
	3	12.689	A

Note: Means with the same letter are not significantly different ($\alpha = 0.05$)

1 inch, the specimen heights of 2 and 3 inches produced the same permeability and slightly higher than that produced by the specimen height of $1\frac{1}{2}$ inches.

Ideally, as the specimen height increases, the effect of aggregate size on the variability of the permeability test result decreases and the reliability of the test result increases. However, the testing time increases as the specimen height increases. Thus, the optimum specimen height should be the smallest one which could still produce reliable results. The results of the Duncan's test indicate that for a maximum aggregate size of 1 inch, the smallest specimen height which could be used is 2 inches. Thus, a specimen height of 2 inches is used in this study.

CHAPTER 4 RESEARCH TESTING PROGRAM

This chapter presents the design of the experiment and the testing procedures used in the laboratory and field testing program in this study.

4.1 Laboratory Testing Program

4.1.1 Design of the Experiment

The purpose of the laboratory testing program is to determine the modulus of rupture and permeability of Florida Classes II, III and IV concretes and the effects of aggregate type, maximum aggregate size, cement type, pozzolan and curing condition on them. A partial factorial design incorporating the following variables is used:

1. Three concrete classes--Florida Classes II, III, & IV
2. Three aggregate types--Brooksville aggregate (Florida porous limestone), a river gravel and Calera aggregate (Alabama dense limestone)
3. Three cement types--Types I, II & III
4. Two pozzolans--Fly ash and silica fume
5. Three maximum aggregate sizes--1 inch, $\frac{3}{4}$ inch and $\frac{3}{8}$ inch
6. Six curing conditions:
 - (1) 7-day moist curing
 - (2) 28-day moist curing
 - (3) 90-day moist curing

- (4) 3-day moist curing followed by 25-day air curing
- (5) 1 day in forms followed by stripping and applying curing compound and 27-day curing
- (6) 16-hour steam curing at 160°F.

The concrete mixes which were evaluated in this laboratory study are displayed in Table 4.1. Two replicate batches per mix were made in order to have a good statistical base for analysis. This amounts to a total of 48 batches.

Due to the increasing importance of the use of silica fume in structural concrete, an additional laboratory study on the effects of silica fume on concrete was conducted. Class IV concrete using silica fume as an addition was tested. Eight batches of concrete using Brooksville aggregate and two batches of concrete using Calera aggregate were made. For the concrete mixes using Brooksville aggregate, the following silica fume additions were used:

- 1) 5% (by weight of cement)
- 2) 10%
- 3) 15%
- 4) 20%.

Two replicate batches per mix design were made for the concrete mixes using Brooksville aggregate. For the concrete mixes using Calera aggregate, the silica fume additions were 10% and 20% and one batch per mix was used. This amounts to a total of 10 batches. The total amounts for the laboratory testing program were 58 batches.

The order of testing the 58 batches of concrete to be evaluated was completely randomized in order to minimize the effects of experimental errors on the test results. These experimental errors might be caused

Table 4.1 Concrete Mixes Evaluated in the Main Laboratory Study

Aggregate Type	Maximum Aggregate Size (inches)	Cement Type	Concrete Class		
			II	III	IV
Brooksville	3/4	I	X	X	X
		II	X	X	X
		II & FLY ASH	X	X	X
		III	X	X	X
	1	I			X
	3/8	I			X
River Gravel	3/4	I	X	X	X
		II			X
		III			X
Calera	3/4	I	X	X	X
		II			X
		III			X

Note: X - two replicates per cell

Total number of batches: 48

by changes in the laboratory environment (changes in temperature and humidity from one season to another), changes in the experience of the testing personnel, or changes in the properties of the materials used. When an experiment is randomized, the experimental errors tend to be more evenly distributed throughout the data collected, and thus, the effects of these possible experimental errors on any single variable are minimized.

The w/c-ratios, cement and pozzolan contents used for the various concrete classes are displayed in Table 4.2. A target slump of 3 inches was aimed for and was achieved by adding appropriate amounts of super-plasticizer to the mixes.

The mix proportions for the concrete mixes used in the laboratory study were determined by using a computer program COMIX [50], which was developed at the University of Florida and was based on the ACI Recommended Practice 211.1 [51]. Trial batches of the designed mixes were made and tested to ensure that the specified requirements (such as slump and strength) could be met. Adjustments to the mix proportions were to be made if necessary.

The trial batches were all prepared in 3.2 cubic foot batches. For each trial batch, eight 6-inch x 12-inch cylindrical specimens and two 6-inch x 6-inch x 30-inch beam specimens were cast. The specimens in the molds were covered with plastic sheets for 20 to 24 hours before they were removed from the molds and cured in the moist room. Two cylinders were tested for compressive strength and two cylinders were tested for splitting tensile strength at 7 days and 28 days. Two beams were tested for modulus of rupture at 28 days.

Table 4.2 Water/Cement-Ratio and Cement Content of
Concrete Mixes in the Laboratory Study

Concrete Class	W/(C + Pozzolan)	Cement (lbs/cy)	Pozzolan (lbs/cy)
II	0.45	564	-
II with Pozzolan	0.45	451	113
III	0.38	658	-
III with Pozzolan	0.38	526	132
IV	0.33	752	-
IV with Pozzolan	0.33	602	150
IV + 5% silica fume	0.33	752	38
IV + 10% silica fume	0.33	752	75
IV + 15% silica fume	0.33	752	113
IV + 20% silica fume	0.33	752	150

A total of nine trial batches were tested. A Type II cement was used in all of the trial batches. The amount of RD-1 (a water-reducing admixture) added to each batch was adjusted until a target slump of 3 inches was reached. Table A.1 in Appendix A displays the mix proportions of these nine trial batches. It can be seen that these nine trial batches cover the three concrete classes (Florida Classes II, III, and IV), the three aggregate types (Brooksville, Calera and river gravel), the three maximum aggregate sizes ($\frac{3}{8}$, $\frac{3}{4}$ and 1 inch) and the two pozzolans (fly ash and silica fume), which are the variables to be evaluated in the main laboratory study.

All trial batches were found to have good workability. A target slump of 3 inches was easily obtained by addition of a very small amount of RD-1 water-reducing admixtures. A slightly larger amount of RD-1 was needed for the Type IV concrete mix (trial batch no. 9), due to its low w/c-ratio.

Table A.2 displays the properties of the fresh concrete and the hardened concrete for the nine trial batches. The air content of most of the mixes ranges from 2.5% to 3.25%, with the exception of the silica fume mix (trial batch no. 7) which has a much lower air content (1.75%). It can be noted from Table A.2 that all of the mixes meet the strength requirements in the FDOT specifications [52]. (The required minimum 28-day compressive strengths for Florida Classes II, III and IV concretes are 3400, 5000 and 5500 psi, respectively.) Thus, no adjustment in the mix proportions was necessary.

The tests performed on each batch of the main laboratory study and concrete with silica fume are displayed in Tables 4.3 and 4.4, respectively. The tests performed on the specimens at the various curing

Table 4.3 Tests Performed on Each Batch of Concrete in the Main Laboratory Study

Curing Condition	Test			
	Beam	Splitting Tensile	Compressive Strength	Permeability
7-day moist curing	X	X	X	
28-day moist curing	⊙	X	⊠	X
90-day moist curing	⊙	X	⊠	X
3-day moist curing & 25-day air curing	X	X	X	X
1 day in form & 27-day curing compound	X	X	X	X
16-hour steam curing	X	X	X	X

Note: X - Two specimens per cell

⊙ - Elastic modulus measured by means of strain gages

⊠ - Elastic modulus measured by means of a compressometer

Schmidt hammer test was performed on all the specimens before test

Table 4.4 Tests Performed on Each Batch of the Concrete with Silica Fume

Curing Condition	Test			
	Beam	Splitting Tensile	Compressive Strength	Permeability
28-day moist curing	(X)	X	[X]	X
90-day moist curing	(X)	X	[X]	X

Note: X - Two specimens per cell

○ - Elastic modulus measured by means of strain gages

□ - Elastic modulus measured by means of a compressometer

Schmidt hammer test was performed on all the specimens before test

conditions included (1) beam test to determine the modulus of rupture and modulus of elasticity, (2) splitting tensile strength test to determine the indirect tensile strength, (3) compressive strength test to determine the compressive strength and modulus of elasticity, and (4) permeability test. Schmidt hammer test was performed on all the specimens before these tests were run.

For each batch of concrete, the following tests were run on the fresh concrete:

1. Slump test (ASTM C143)
2. Air content test (ASTM C173)
3. Unit weight test (ASTM C138)

4.1.2 Fabrication of Concrete Specimens

4.1.2.1 Mixing of concrete. The concrete batches used in the laboratory testing program were mixed in a rotary type mixer with a maximum capacity of 25 cubic feet. A total of 10 batches (12 cubic feet each) of concrete with silica fume and 48 batches (17 cubic feet each) of the concrete for the main laboratory study were prepared. The aggregates and cement were added and mixed for 3 minutes with approximately 80% of the required mixing water. After 3 minutes, the mixer was shut off and the concrete allowed to rest for 2 minutes. Thereafter, the remaining water was added to the mixture and mixing was continued for 2 more minutes. A slump test was then run to determine if a target slump of 3 inches had been reached. If the slump was too low, a water reducing admixture, Mighty RD-1 was added to the mixture, which was mixed for 2 minutes more. Another slump test was run. This process was repeated until the slump was reached.

4.1.2.2 Casting of concrete specimens. After mixing was completed, the concrete was discharged into a large hopper, from which concrete was extracted as needed for fresh concrete tests, and for casting the beams and cylinders. The following tests were performed on the fresh concrete: slump test, air content test and unit weight test. The concrete mixture was then placed in 6-inch x 12-inch cardboard cylinder molds, 4-inch x 8-inch cardboard cylinder molds and 6-inch x 6-inch x 30-inch steel beam molds. The molds were placed on a vibrating table and vibrated while the cylinders or beams were being filled. For each batch of concrete in the main laboratory study, twelve 6-inch x 6-inch x 30-inch beam specimens, twenty-four 6-inch x 12-inch cylindrical specimens and ten 4-inch x 8-inch cylindrical specimens were cast. Also, for the ten batches of concrete with silica fume, four 6-inch x 6-inch x 30-inch beam specimens, eight 6-inch x 12-inch cylindrical specimens and four 4-inch x 8-inch cylindrical specimens were cast. All of these specimens were cured under a plastic cover for about 20 to 24 hours before they were removed from the molds and subjected to various curing conditions.

4.1.2.3 Curing of concrete specimens. The six curing conditions which are listed in Section 4.1 were used on the concrete specimens. For the first three curing conditions, namely 7-day, 28-day and 90-day moist curing, the designated specimens were cured in a moist room for 6, 27 and 89 days, respectively, after they were removed from the molds. The moist room conforms to the ASTM requirements of the greater than 98% relative humidity and $73 \pm 3^{\circ}\text{F}$ temperature.

For the fourth curing condition, the specimens, after removal from the molds, were cured in the moist room for two days. The specimens

were then removed from the moist room and allowed to cure in room environment for 25 days before testing.

For the fifth curing condition, the specimens were coated with a white-pigment curing compound, #64RB RESIN CURE-WHITE, which was applied to all surfaces of the specimens in two layers. These specimens were then left to cure for 27 more days in room environment before testing.

The sixth curing condition was a 16-hour steam curing at 160°F. The specimens, after removal from the molds, were placed into a steam oven which was initially at room temperature. The temperature of the steam oven was then gradually increased to 160°F in two hours. The steam oven was then maintained at 160°F for 12 hours. The steam oven was then allowed to cool slowly down to room temperature in two hours before the specimens were removed from the steam oven for testing. Figure 4.1 shows the steam generator and steam oven, which were used to provide this curing condition.

4.1.3 Testing of Concrete Specimens

The tests performed on the cured concrete specimens were permeability, flexural strength, compressive strength, splitting tensile strength and Schmidt hammer tests. The tests to be performed on the fresh concrete were slump, air content and unit weight tests. While the permeability test has been described in Chapter 3, the rest of the tests are described in this section.

4.1.3.1 Modulus of rupture or flexural strength test. Flexural strength tests were performed in accordance with the ASTM Standard Test Method C78 for Flexural Strength of Concrete Using Simple Beam with Third-Point Loading [43]. Figure 4.2 shows the test set-up for this



Figure 4.1 The Steam Generator and Steam Oven

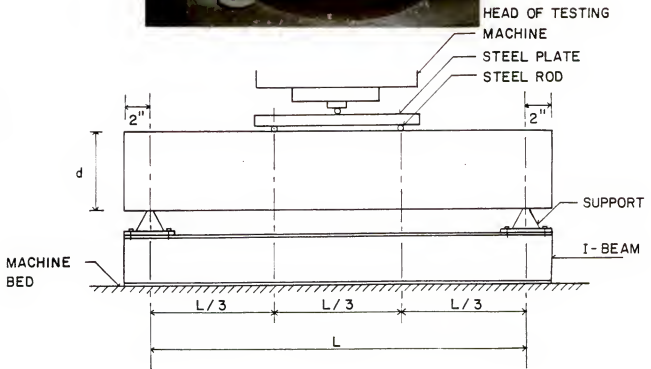


Figure 4.2 Set-Up for Flexural Test of Concrete by Third-Point Loading Method

test. Beam specimens measuring 6 inches x 6 inches x 30 inches were used. The modulus of rupture was calculated as follows:

$$f_r = \frac{PL}{bd^2} \quad (4.1)$$

where f_r = flexural strength or modulus of rupture, in psi

P = maximum applied load, in lbs.

L = span length in inches

b = average width of specimen, in inches

d = average depth of specimen, in inches.

When fracture occurs outside of the middle third of the span length by not greater than 5% of the span length, the following equation was used to calculate the modulus of rupture:

$$f_r = \frac{3Pa}{bd^2} \quad (4.2)$$

where a = average distance between line of fracture and the nearest support measured on the tension surface of the beam, in inches.

If failure occurs outside of the middle third of the span length by more than 5%, the test results were discarded.

Elastic modulus of concrete was determined from the specimens which had been moist cured for 28 and 90 days. Two half-inch strain gages (Gage Type EA-06-500 BH-120 by Micro-Measurements) were adhered to the bottom side and at the center of each beam specimen. Figure 4.3 shows the location of the strain gages on the beam specimen. The strain gages were placed in the longitudinal direction and at approximately 1 inch from the edge of the beam, as shown in the figure. The strain gages

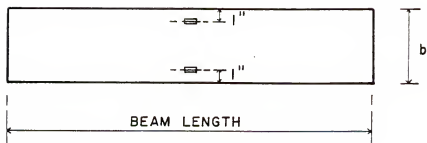


Figure 4.3 Locations of Strain Gages on Concrete Beam Specimens

were connected to and calibrated with a Vishay/Ellis strain indicator system. Strain readings from the two strain gages were recorded at every 500-pound load interval, as the beam was loaded in the flexural strength test. The means of the two strain readings were used to plot the stress-strain diagrams which were used to determine the modulus of elasticity. The slope of the line from the origin to the stress-strain plot at 40% of the modulus of rupture was taken to be the modulus of elasticity.

4.1.3.2 Compressive strength test. Compressive strength tests were performed in accordance with the ASTM Standard Test Method C39-83b for Compressive Strength of Cylindrical Concrete Specimens [43]. Cylindrical specimens measuring 6 inches x 12 inches were used for this test.

Concrete specimens which had been moist cured for 28 and 90 days were also evaluated for their moduli of elasticity. A compressometer was used to measure the deformation of the cylindrical specimen as it was loaded in the compressive strength test. The ASTM Standard Method C469-83 for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression was followed with a slight modification and improvement. The test set-up is shown in Figure 4.4. The deformation of the cylindrical specimen was recorded at every 3-kip load interval up to about 40% of the ultimate compressive strength. The stress-strain diagram was plotted. The best-fitted line through the stress-strain plot was drawn. If the best-fitted line did not pass through the origin, it would be shifted to the origin. The slope of the stress-strain curve was used to determine the modulus of elasticity.

4.1.3.3 Splitting tensile strength test. Splitting tensile strength tests were performed in accordance with ASTM Standard Method



Figure 4.4 Test Set-Up for Measuring the Modulus of Elasticity of Cylindrical Concrete Specimens

C496 for Splitting Tensile Strength of Cylindrical Concrete Specimens [43]. Cylindrical specimens measuring 6 inches x 12 inches were used for this test. Figure 4.5 shows the set-up for this test. The splitting tensile strength was computed as follows:

$$f_{ct} = \frac{2P}{\pi ld} \quad (4.3)$$

where f_{ct} = splitting tensile strength, in psi

P = maximum applied load, in lbs

l = length of cylinder, in inches

d = diameter of cylinder, in inches.

4.1.3.4 Schmidt hammer test. Schmidt hammer tests were run on all the specimens before they were failed in the various strength tests. The Schmidt hammer was placed in the horizontal position and perpendicular to the surface of the concrete specimens to obtain the rebound readings. Six rebound readings were obtained from each specimen.

4.1.3.5 Tests on fresh concrete. Slump, air content and unit weight tests were performed on the fresh concrete before the specimens were cast. Slump test was performed in accordance with ASTM Standard Method C143 for Slump of Portland Cement Concrete. ASTM Standard Method C173 for Air Content of Freshly Mixed Concrete by the Volumetric Method was used to determine the air content. Unit weight was determined in accordance with ASTM Standard Method C138. A measure with a volume of 0.5 cubic foot was used for running the unit weight test.

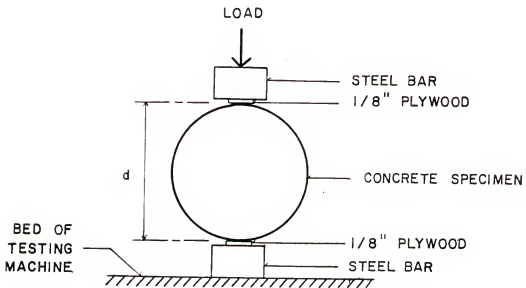


Figure 4.5 Set-Up for Splitting Tensile Strength Test

4.2 Field Testing Program

Properties of in-service Florida concretes were evaluated and compared to those of the laboratory-cured concretes. Two projects for each of the three classes of concrete (Classes II, III, & IV) were selected and used. For each project, fresh concrete was collected and placed in six 6-inch x 6-inch x 30-inch beam molds, twelve 6-inch x 12-inch cylindrical molds, and four 4-inch x 8-inch cylindrical molds at the job site. The following tests were run on the fresh concrete:

- a) Slump test
- b) Air content test
- c) Unit weight test.

The quantity of fresh concrete required for the specimens and tests amounts to 8.0 cu. ft. per project.

The concretes in the molds were consolidated to a comparable field density. The following information were obtained on each concrete mix:

- a) Mix proportions
- b) Properties of aggregate
- c) Properties of portland cement
- d) Properties of admixtures
- e) Air temperature and mix temperature at time of placement
- f) Air humidity at time of placement.

The concrete specimens in the molds were sealed with plastic sheets and transported to the moist room in the Department of Civil Engineering, which conforms to the ASTM requirements of greater than 98% relative humidity and $73^{\circ} \pm 3^{\circ}\text{F}$ temperature. The molds were removed after two days and the specimens were to continue to be cured in the moist room until the specified test time.

The tests performed on the concrete specimens from each project are shown in Table 4.5. Two replicate specimens were evaluated in each of the tests. The beam, splitting tensile and compressive strength tests were run at 7 days, 28 days, and 90 days. The permeability tests were performed after 28 days and 90 days curing time. Strain gages were attached to the beam specimens tested at 28 days and 90 days to monitor the strains during loading and to determine the elastic modulus. A compressometer was used to determine elastic modulus of the cylindrical specimen as it was loaded in the compressive strength test at 28 days and 90 days. Schmidt hammer tests were performed on all the specimens before these tests were run.

Six cored samples (4-inch diameter) were obtained from each of these six project sites at 28 days after placement. Two samples were evaluated in each of the following tests: compressive test, splitting tensile test and the permeability test.

Table 4.5 Experimental Design for Testing of Specimens from Each Project

Curing Time	Test			
	Beam	Splitting Tensile	Compressive Strength	Permeability
7 days	X	X	X	
28 days	⊙	X	⊠	X
90 days	⊙	X	⊠	X

Note: X - Two specimens per cell

⊙ - Elastic modulus measured by means of strain gages

⊠ - Elastic modulus measured by means of compressometer

Schmidt hammer test was performed on all the specimens before test

CHAPTER 5 MATERIALS

This chapter presents the properties of the portland cements, pozzolans, aggregates, admixtures and curing compound used in the laboratory testing program of this study.

5.1 Cements

The cements used were Types I, II and III portland cements manufactured by General Portland, Inc., Tampa, Florida. The results of physical and chemical analyses on these three cements were supplied by the manufacturer and are listed in Tables 5.1 and 5.2.

5.2 Pozzolans

The two pozzolans used were a fly ash and a silica fume. Fly ash was supplied by Florida Mining and Materials Corporation and classified as Class F, according to ASTM C618. Detailed information on the chemical and physical properties of fly ash used are listed in Tables 5.1 and 5.2. The silica fume used (Force 10000) was in a slurry form and had a specific gravity of 2.24. It was supplied by W. R. Grace and Company.

5.3 Aggregate

Three types of coarse aggregate were used. They were a Florida limestone (Brooksville aggregate), an Alabama limestone (Calera aggregate) and a river gravel. Three maximum aggregate sizes were used for

Table 5.1 Results of Physical Tests on Cement and Fly Ash Used

	Type I Cement	Type II Cement	Type II Cement	Fly Ash
Fineness by Blaine air permeability test, cm ² /g	3970	3970	6410	-
Soundness (autoclave expansion), percent	0.09	-0.01	0.02	0.14
Time of setting (Gilmore):				
Initial, hrs: mins	2:15	2:45	2:05	2:50
Final, hrs: mins	4:05	4:25	3:55	4:40
Compressive Strength:				
1 Day, psi	2230	1990	3960	
3 Days, psi	3600	3040	5290	
7 Days, psi	4810	4190	6450	
Air Entrainment, percent	9.5	9.1	6.3	

Table 5.2 Results of Chemical Analyses on Cements and Fly Ash Used

	Type I Cement	Type II Cement	Type III Cement	Fly Ash
Silicon Dioxide (SiO_2), percent	20.5	21.6	20.1	} 88.16
Aluminum Oxide (Al_2O_3), percent	5.1	4.5	5.0	
Ferric Oxide (Fe_2O_3), percent	3.0	4.1	3.0	
Magnesium Oxide (MgO), percent	1.0	0.6	1.0	
Sulfur Trioxide (SO_3), percent	3.3	2.9	4.2	1.8
Loss on Ignition, percent	1.4	0.9	1.9	3.0
Insoluble Residue, percent	0.18	0.17	0.21	
Alkalis ($\%\text{Na}_2\text{O}+0.658\text{K}_2\text{O}$), percent	0.35	0.36	0.33	
Tricalcium Silicate, percent	61	54	58	
Dicalcium Silicate, percent	13	21	14	
Tricalcium Aluminate, percent	8.5	4.8	8.4	
Tetracalcium Alumino Ferrite, percent	9.3	12.6	9.0	

the Brookville aggregate. They were #57, #67 and #89 stones with maximum sizes of 1, $\frac{3}{4}$ and $\frac{3}{8}$ inch, respectively. For the Calera aggregate and the river gravel, only one maximum aggregate size, namely $\frac{3}{4}$ inch, was used. The physical properties of the coarse aggregates are displayed in Table 5.3. The gradation charts of the Brooksville #57, Brooksville #67, Brooksville #89, Calera and river gravel aggregates from first truck loads are shown in Figures 5.1 through 5.5, respectively.

The fine aggregate used for all mixtures as a fine sand from Goldhead, Florida. The physical properties of this sand from all truck loads are displayed in Table 5.4. The gradation of this sand from truck load no. 1 is depicted in Figure 5.6.

5.4 Admixtures and Curing Compound

An admixture, Mighty RD-1 was used to adjust the slump of the fresh concrete to a target slump of 3 inches. This admixture meets all the requirements of ASTM C494 Type G and Type D, and is classified as a water-reducer and retarder.

A curing compound, #64 RB RESIN CURE, was used to provide one of the curing conditions in the laboratory study. It meets the requirements of ASTM C309-81 specifications.

Table 5.3 Physical Properties of Coarse Aggregates

Coarse Aggregate	Truck Load No.	Properties of Coarse Aggregate				
		Bulk Specific Gravity (Dry)	Bulk Specific Gravity (SSD)	Absorption (percent)	Unit Weight (lbs/ft ³)	Los Angeles Abrasion Ratio (percent)
Brooksville #57	1	2.37	2.45	3.24	91.0	-
Brooksville #67	1	2.39	2.45	3.16	92.0	34
	2	2.35	2.43	3.52	88.0	38
Brooksville #89	1	2.43	2.50	2.95	92.0	-
Calera #67	1	2.69	2.70	0.46	98.0	-
	2	2.75	2.76	0.38	102.0	17
River Gravel #67	1	2.57	2.60	1.13	102.0	-
	2	2.58	2.604	0.885	102.0	37

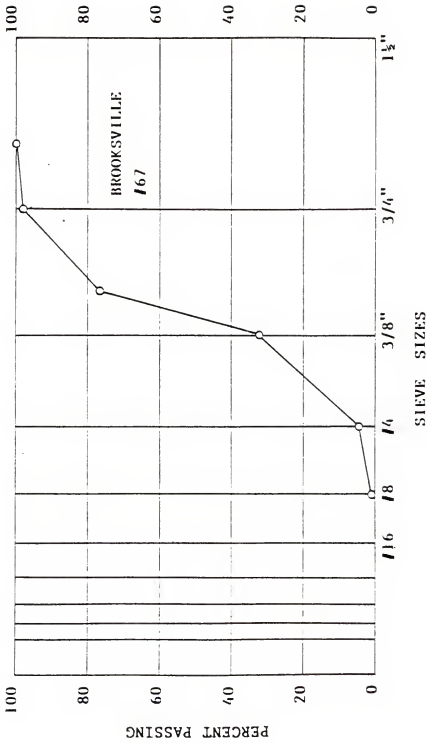


Figure 5.2 Gradation Chart for Brooksville #67 Coarse Aggregate

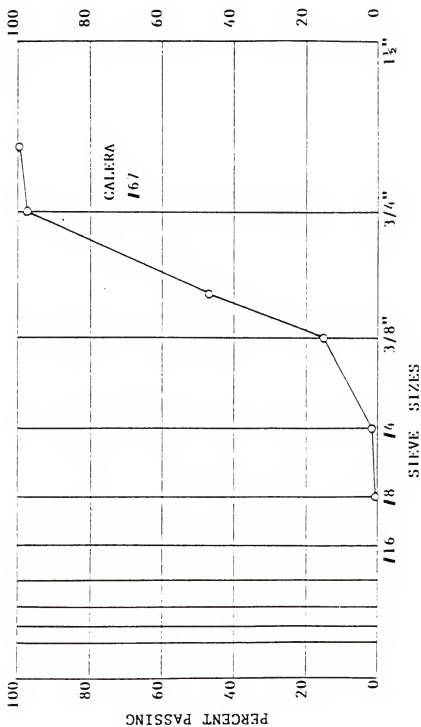


Figure 5.4 Gradation Chart for Calera Coarse Aggregate

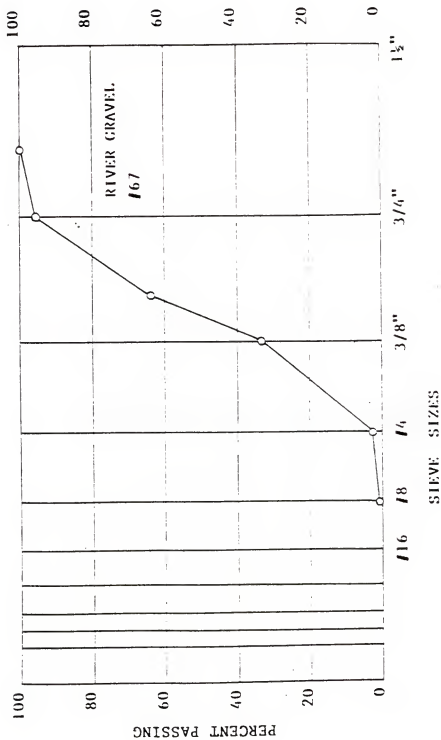


Figure 5.5 Gradation Chart for the River Gravel Coarse Aggregate

Table 5.4 Physical Properties of Fine Aggregate

Truck Load No.	Properties of Fine Aggregate			
	Bulk Specific Gravity (Dry)	Bulk Specific Gravity (SSD)	Absorption (percent)	Fineness Modulus
1	2.54	2.55	0.75	2.15
2	2.51	2.52	0.68	2.36
3	2.53	2.54	0.70	2.24
4	2.52	2.53	0.70	2.33
5	2.57	2.58	0.64	2.27
6	2.63	2.65	0.66	2.28
7	2.60	2.62	0.95	2.28

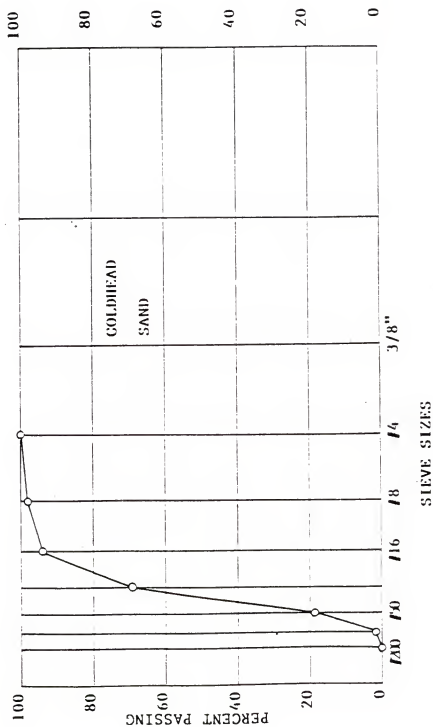


Figure 5.6 Gradation Chart for Goldhead Sand

CHAPTER 6 METHOD OF ANALYSIS OF DATA

6.1 Introduction

The purpose of the laboratory testing program was to evaluate the effects of aggregate type, aggregate size, cement type, w/c-ratio, fly ash, silica fume and curing condition on the permeability, modulus of rupture, compressive strength and splitting tensile strength of structural concretes in Florida. The evaluation of the effects of these various factors (mix parameters and curing conditions) on the properties (permeability and strengths) of the concrete studied was aided by using the analysis of variance (ANOVA) statistical method. Since the testing program was a partial factorial experiment, the data could not be analyzed by using one ANOVA model which incorporates all the factors studied. However, the testing program could be perceived as consisting of six smaller full factorial experiments, each of which incorporates certain factors. Thus, the test results were analyzed as if there were six separate sets of experiments.

This chapter presents the description of the method of analysis, and the six experimental designs and the corresponding ANOVA models.

6.2 Method of Analysis

The ANOVA was used to determine whether the effects of the main factors and interactions of factors were statistically significant. The most important results of an ANOVA are the P-values for the factors

considered. Basically, the P-value for a factor indicates the probability of error of the statement that the factor has a significant effect on the measured parameter. For example, if a factor A has a P value of 0.05, the statement that factor A is significant will have a probability of error of 0.05. A lower P value indicates a higher level of significance. In this study, a probability of error (α level) of 0.05 was used. If the P value of a factor is equal to or less than 0.05, the factor is considered to be significant.

Before an ANOVA was performed, the assumption of homogeneity of variance was checked using the BURR-FOSTER Q-Test [53]. If the Q-test value was less than the Q-critical value at $\alpha = 0.001$, the assumption of homogeneity of variance would be accepted. If it was rejected, an appropriate transformation would be applied to the variable, and the transformed variable would be used in the ANOVA. The results of all the BURR-FOSTER Q-tests performed can be found in Appendix D.

The Duncan's multiple-range test was used to compare the means of concrete properties (such as modulus of rupture and permeability of concrete) at different curing conditions, aggregate type, cement type maximum aggregate size, and w/c-ratios. A level of significance of 0.05 ($\alpha = 0.05$) was also used to compare the means in this study.

ANOVA and Duncan's multiple-range test were performed on the test data using SAS computer software, which is available at the Center for Instructional and Research Computing Activities (CIRCA), University of Florida.

6.3 Experimental Designs

6.3.1 Design No. 1

The effects of aggregate type, cement type, and curing condition on the modulus of rupture, compressive strength, splitting tensile strength, and permeability of Florida concrete were evaluated at fixed cement content of 752 lbs/yd³, w/c-ratio of 0.33, and maximum aggregate size of 3/4 inch (aggregate #67). The test results were analyzed as a result of a full factorial experiment comprising of three types of aggregates and three types of cement. There were six curing conditions for each batch. The mix combinations and the batch numbers for this experimental design are displayed in Table 6.1. ANOVA was carried out by using the following linear model:

$$Y_{ijk\ell m} = \mu + A_i + C_j + (AC)_{ij} + B_{(ij)k} + \delta_{(ijk)} + D_{\ell} + (AD)_{i\ell} \\ + (CD)_{j\ell} + (ACD)_{ij\ell} + (BD)_{(ij)k\ell} + \epsilon_{(ijk\ell)m} \quad (6.1)$$

$$i = j = 1, 2, 3; \quad k = m = 1, 2; \quad \ell = 1, 2, 3, 4, 5, 6$$

where $Y_{ijk\ell m}$ = the response of m^{th} specimen of ℓ^{th} curing condition,
 k^{th} batch, j^{th} cement type and i^{th} aggregate type

μ = overall mean

A_i = effect of the i^{th} aggregate type

C_j = effect of the j^{th} cement type

$(AC)_{ij}$ = effect of the interaction of the i^{th} aggregate type and
the j^{th} cement type

$B_{(ij)k}$ = effect of the k^{th} batch in i^{th} aggregate type and j^{th}
cement type

Table 6.1 Experimental Design for Study on the Effects of Aggregate Type, Cement Type and Curing Condition (Design No. 1)

Aggregate	Cement Type		
	I	II	III
Brooksville	B #10	B #6	B #5
	B #41	B #44	B#28
Calera	B #21	B #13	B #15
	B #38	B #40	B #37
River Gravel	B #25	B #17	B #23
	B #47	B #35	B #30

Note: W/C-ratio of 0.33 and maximum aggregate size of $\frac{3}{4}$ inch were used for all batches

There were six curing conditions for each batch

$\delta_{(ijk)}$	= restriction error
D_ℓ	= effect of the ℓ^{th} curing condition
$(AD)_{i\ell}$	= effect of the interaction of the i^{th} aggregate type and the ℓ^{th} curing condition
$(CD)_{j\ell}$	= effect of the interaction of the j^{th} cement type and the ℓ^{th} curing condition
$(ACD)_{ij\ell}$	= effect of the interaction of the i^{th} aggregate type, j^{th} cement type and ℓ^{th} curing condition
$(BD)_{(ij)k\ell}$	= effect of the interaction of the ℓ^{th} curing condition and k^{th} batch in i^{th} aggregate type, j^{th} cement type
$\varepsilon_{(ijk\ell)m}$	= random error.

The restriction error, δ , as was due to the fact that the specimens which were subjected to different curing conditions were restricted to the same batch of concrete. Table 6.2 displays the ANOVA table for this model, showing the degree of freedom (df) and the expected mean square (EMS) of all terms in the model. The interaction term B(AC) was used to test the main effects of aggregate type (A) and cement type (C) and the effect of interaction AC. The interaction term BD(AC) was used to test the effect of curing condition (D) and the interactions of AD, CD, and ACD. The error term (ε) was used to test the interaction BD(AC). Since the interaction BD(AC) is a high order interaction term, it usually would not give any important information. Therefore, it was not fully evaluated in this study.

6.3.2 Design No. 2

The second experimental design was used to evaluate the effects of aggregate type, concrete class (w/c-ratio), and curing condition on the

Table 6.2 ANOVA Table for Design No. 1

Sources	df	EMS
A	2	$\sigma^2 + 12\sigma_\delta^2 + 12\sigma_b^2 + 72\sigma_a^2$
C	2	$\sigma^2 + 12\sigma_\delta^2 + 12\sigma_b^2 + 72\sigma_c^2$
AC	4	$\sigma^2 + 12\sigma_\delta^2 + 12\sigma_b^2 + 24\sigma_{ac}^2$
B(AC)	9	$\sigma^2 + 12\sigma_\delta^2 + 12\sigma_b^2$
δ	0	$\sigma^2 + 12\sigma_\delta^2$
D	5	$\sigma^2 + 2\sigma_{bc}^2 + 36\sigma_d^2$
AD	10	$\sigma^2 + 2\sigma_{bc}^2 + 12\sigma_{ad}^2$
CD	10	$\sigma^2 + 2\sigma_{bc}^2 + 12\sigma_{cd}^2$
ACD	20	$\sigma^2 + 2\sigma_{bc}^2 + 4\sigma_{acd}^2$
BD(AC)	45	$\sigma^2 + 2\sigma_{bc}^2$
ϵ	108	σ^2

strengths and permeability of Florida concrete. The experimental design is shown in Table 6.3. The factors included were three types of aggregate and three w/c-ratios. There were two replicate batches for each mix design and six curing conditions for each batch. All batches in this design were made with cement Type I and aggregate #67 (maximum aggregate size = $3/4$ inch).

The statistical model for Design No. 2 is similar to the one used in Design No. 1, and is shown below:

$$\begin{aligned}
 Y_{ijk\ell m} = & \mu + A_i + W_j + (AW)_{ij} + B_{(ij)k} + \delta_{(ijk)} + D_{\ell} + (AD)_{i\ell} \\
 & + (WD)_{j\ell} + (AWD)_{ij\ell} + (BD)_{(ij)k\ell} + \varepsilon_{(ijk\ell)m}
 \end{aligned} \quad (6.2)$$

$$i = j = 1, 2, 3; \quad k = m = 1, 2; \quad \ell = 1, 2, 3, 4, 5, 6$$

where W_j = effect of the j^{th} w/c-ratio, and

μ , Y , A , B , D , and ε are as defined in Equation 6.1.

6.3.3 Design No. 3

In Design No. 3, the effects of the w/c-ratio, cement type and curing condition on the strengths and permeability of Florida concrete were studied at fixed aggregate type (Brooksville aggregate) and maximum aggregate size of $3/4$ inch. The design is presented in Table 6.4. The factors included were three w/c-ratios and three types of cement. Each mix design was replicated twice. Six curing conditions were used for each batch.

Table 6.3 Experimental Design for Study on the Effects of Aggregate Type, W/C-Ratio and Curing Condition (Design No. 2)

Aggregate Type	W/C-Ratio		
	0.33	0.38	0.45
Brooksville	B #10	B #2	B #3
	B #41	B #42	B #29
Calera	B #21	B #18	B #9
	B #38	B #27	B #33
River Gravel	B #25	B #20	B #8
	B #47	B #43	B #31

Note: Cement Type I and maximum aggregate size of $\frac{3}{4}$ inch were used for all batches.

There were six curing conditions for each batch.

Table 6.4 Experimental Design for Study on the Effects of W/C-Ratio, Cement Type and Curing Condition (Design No. 3)

W/C-Ratio	Cement Type		
	I	II	III
0.33	B #10	B #6	B #5
	B #41	B #44	B #28
0.38	B #2	B #4	B #14
	B #42	B #45	B #36
0.45	B #3	B #1	B #16
	B #29	B #39	B #22

Note: Brooksville #67 was used for all batches.

There were six curing conditions for each batch.

The statistical model for this design is similar to the one used in design no. 1 and is shown below

$$Y_{ijk\ell m} = \mu + W_i + C_j + (WC)_{ij} + B_{(ij)k} + \delta_{(ijk)} + D_{\ell} + (WD)_{i\ell} \\ + (CD)_{j\ell} + (WCD)_{ij\ell} + (BD)_{(ij)k\ell} + \epsilon_{(ijk\ell)m} \quad (6.3)$$

$$i = j = 1, 2, 3; \quad k = m = 1, 2; \quad \ell = 1, 2, 3, 4, 5, 6$$

The terms in the model are as defined in Equations 6.1 and 6.2.

6.3.4 Design No. 4 (Fly Ash Concrete)

The effects of fly ash, w/c-ratio and curing condition on the strengths and permeability of Florida concrete were analyzed at fixed aggregate type (Brooksville aggregate), cement type (Type II) and maximum aggregate of $3/4$ inch. The experimental design is displayed in Table 6.5. The factors included were two levels of percent replacement of cement by fly ash and three w/c-ratios. Two replicate batches per mix design were prepared. Each batch consisted of six curing conditions.

The statistical model for this design is similar to the one used in Design No. 1 and is shown below:

$$Y_{ijk\ell m} = \mu + F_i + W_j + (FW)_{ij} + B_{(ij)k} + \delta_{(ijk)} + D_{\ell} + (FD)_{i\ell} \\ + (WD)_{j\ell} + (FWD)_{ij\ell} + (BD)_{(ij)k\ell} + \epsilon_{(ijk\ell)m} \quad (6.4)$$

$$i = k = m = 1, 2; \quad j = 1, 2, 3; \quad \ell = 1, 2, 3, 4, 5, 6$$

where F_i = effect of the i^{th} level of replacement of cement by fly ash, and the other terms are as defined in Equations 6.1 and 6.2.

Table 6.5 Experimental Design for Study on the Effects of Fly Ash, W/C-Ratio and Curing Condition (Design No. 4)

Percent of Fly Ash	W/C-Ratio		
	0.33	0.38	0.45
0	B #6	B #4	B #1
	B #44	B #45	B #39
20	B #24	B #26	B #19
	B #46	B #32	B #48

Note: Brooksville #67 and cement Type II were used for all batches.
There were six curing conditions for each batch.

6.3.5 Design No. 5

In Design No. 5, the effects of the maximum aggregate size and curing condition on the strengths and permeability of Florida concrete were studied. The experimental design is presented in Table 6.6. The factors included were three maximum aggregate sizes and six curing conditions. Two replicate batches were prepared for each mix design. All batches were made at fixed w/c-ratio of 0.33, cement content of 752 lbs/yd³, aggregate type (Brooksville aggregate) and cement type (Type I).

The statistical model for this design is:

$$Y_{ijk\ell} = \mu + M_i + B_{(i)j} + \delta_{(ij)} + D_k + (MD)_{ik} + (BD)_{(i)jk} + \epsilon_{(ijk)\ell} \quad (6.5)$$

$$i = 1, 2, 3; \quad j = \ell = 1, 2; \quad k = 1, 2, 3, 4, 5, 6$$

where M_i = effect of the i^{th} maximum aggregate size, and

μ , B , δ , D and ϵ are defined in Equation 6.1.

Table 6.7 displays the ANOVA table for this model. The error term (ϵ) was used to test the interaction $BD(M)$. The $BD(M)$ was then used to test the effects of curing condition (D) and the interaction of maximum aggregate size by curing condition (MD). The effect of maximum aggregate size was tested by $B(M)$. In this model, if the term $BD(M)$ was determined to be insignificant, it would then be pooled to the error term. The new error term which would have a higher degree of freedom would be used to test the effects of curing condition (D) and the interaction of MD .

Table 6.6 Experimental Design for Study on the Effects of Maximum Aggregate Size and Curing Condition (Design No. 5)

Maximum Aggregate Size		
1 inch	$\frac{3}{4}$ inch	$\frac{3}{8}$ inch
B #12	B #10	B #7
B #34	B #41	B #11

Note: Brooksville aggregate, cement Type I, w/c-ratio of 0.33, and cement content of 752 lbs/yd³ were used for all batches.

There were six curing conditions for each batch.

Table 6.7 ANOVA Table for Design No. 5

Sources	df	EMS
M	2	$\sigma^2 + 12\sigma_\delta^2 + 12\sigma_b^2 + 24\sigma_m^2$
B(M)	3	$\sigma^2 + 12\sigma_\delta^2 + 12\sigma_b^2$
δ	0	$\sigma^2 + 12\sigma_\delta^2$
D	5	$\sigma^2 + 2\sigma_{bd}^2 + 12\sigma_d^2$
MD	10	$\sigma^2 + 2\sigma_{bd}^2 + 4\sigma_{md}^2$
BD(M)	15	$\sigma^2 + 2\sigma_{bd}^2$
ϵ	36	σ^2

6.3.6 Design No. 6 (Silica Fume Concrete)

The effects of percent addition of silica fume and curing condition on the strengths and permeability of Florida concrete were evaluated. The experimental design is displayed in Table 6.8. The factors included were five levels of percent addition of silica fume and two curing conditions. Two batches per mix design were made. The Brooksville aggregate, cement Type II, water-cementitious ratio of 0.33, cement content of 752 lbs/yd³ and maximum aggregate of 3/4 inch were used for all batches.

The statistical model for this design is similar to the one used in design no. 5 and is shown below:

$$Y_{ijk\ell} = \mu + S_i + B_{(i)j} + \delta_{(ij)} + D_k + (SD)_{ik} + BD_{(i)jk} + \epsilon_{(ijk)\ell} \quad (6.6)$$

$$i = 1, 2, 3, 4, 5; \quad j = \ell = k = 1, 2$$

where S_i = effect of the i^{th} percent addition of silica fume, and

μ , B , δ , D and ϵ are as defined in Equation 6.1.

Table 6.8 Experimental Design for Study on the Effects of Percent Addition of Silica Fume and Curing Condition (Design No. 6)

Percent Addition of Silica Fume				
0	5	10	15	20
B #6	B #S1	B #S3	B #S5	B #S7
B #44	B #S2	B #S4	B #S6	B #S8

Note: Brooksville #67, cement Type II, water-cementitious ratio of 0.33, and cement content of 752 lbs/yd³ were used for all batches. There were two curing conditions for all batches.

CHAPTER 7 RESULTS OF MODULUS OF RUPTURE TESTS

7.1 Introduction

The modulus of rupture of concrete is an indicator of its flexural resistance to loads. It has been used particularly for control of concrete mixes for such structures as bridge decks and highway pavements [2]. Very little test data on this property of Florida structural concrete are available at the present time. Therefore, this research program was conducted to determine, among other things, the modulus of rupture of Florida structural concretes and its degree of dependence on mix parameters and curing conditions.

This chapter presents the results of the modulus of rupture tests in all six experimental designs (designs 1 through 6). The details of all the experimental designs are presented in Chapter 6. The general laboratory procedure described in Chapter 4 was used to prepare and test the concrete specimens. The method of analysis of the test results are also described in Chapter 6.

7.2 Analysis of Results

7.2.1 Results of Experimental Design No. 1

This section presents the results of experimental design no. 1, which dealt with the effects of aggregate type, cement type and curing condition on the modulus of rupture. The results of tests on 18 batches from the main laboratory study, which are presented in Table C.1 in

Appendix C, were analyzed. Mix parameters and batch numbers for all these 18 batches can be found in Table 6.1. The mean moduli of rupture of concretes using three different aggregates and cement types at the six curing conditions are presented in Table 7.1.

ANOVA was performed on the test results by using the linear model as shown in Equation 6.1. Results of ANOVA are summarized in Table 7.2. From Table 7.2, it is seen that the effects of aggregate type (A), cement type (C), curing condition (D), and the interaction AD are significant while the interactions AC, CD and ACD are not significant.

The comparison of means for all the main effects were made as follows:

1. Since aggregate type interacts with the curing condition and does not interact with the cement type, the effects of aggregate type can be evaluated by comparing the means of modulus of rupture for three aggregate types, averaged over the cement types at each curing condition.
2. Since cement type does not interact with the other two factors, the effects of cement type can be evaluated by comparing the means for three cement types, averaged over the other two factors.
3. Since curing condition interacts with the aggregate type and does not interact with the cement type, the effects caused by the six curing conditions can be assessed by comparing the means for the six curing conditions averaged over the cement type of each aggregate type.

The Duncan's multiple-range test at a level significance of 0.05 was used to compare the means of modulus of rupture. Table 7.3 shows

Table 7.1 Mean Moduli of Rupture of Concrete in Design No. 1

Aggregate Type	Cement Type	Modulus of Rupture (psi)					
		Curing Condition*					
		1	2	3	4	5	6
Brooksville	I	682	779	821	681	665	809
	II	654	630	735	629	654	779
	III	823	822	864	752	769	867
Calera	I	837	940	967	797	750	1119
	II	815	917	953	689	685	1047
	III	1015	1056	1086	783	764	1164
River Gravel	I	709	745	884	697	694	908
	II	653	724	834	605	622	958
	III	835	923	990	722	705	1092

Note: * Curing conditions:

1 = 16 hours steam curing

2 = 7 days moist curing

3 = 28 days moist curing

4 = 3 days moist curing and 25 days in air

5 = 1 day in form and 27 days with curing compound

6 = 90 days moist curing

Table 7.2 Result of ANOVA on Modulus of Rupture for Design No. 1

Factor	df	MS	F-Value	P
A	2	522781.3	34.80	0.0001*
C	2	346768.2	23.08	0.0003*
AC	4	1993.8	0.13	0.9663
B(AC)	9	15024.2	-	-
D	5	429701.7	89.42	0.0001*
AD	10	31719.4	6.60	0.0001*
CD	10	7544.7	1.57	0.1469
ACD	20	3979.9	0.83	0.6689
BD(AC)	45	4805.5	-	-
ϵ	108	1480.3		

Note: * Significant at α level of 0.05

Table 7.3 Grouping of Aggregate Type by Curing Condition on the Basis of Modulus of Rupture from Duncan's Test (Design No. 1)

Curing Condition*	Aggregate Type	Means (psi)	Grouping
AC	Calera	756	A
	Brooksville	687	A
	River Gravel	675	A
CC	Calera	733	A
	Brooksville	688	A
	River Gravel	673	A
SC	Calera	889	A
	Brooksville	719	B
	River Gravel	732	B
7-MC	Calera	971	A
	Brooksville	744	B
	River Gravel	797	B
28-MC	Calera	1002	A
	Brooksville	807	C
	River Gravel	903	B
90-MC	Calera	1110	A
	Brooksville	818	C
	River Gravel	986	B

Note: - Means with the same letters are not significantly different ($\alpha = 0.05$)

* = Curing Condition

AC = 3 days moist curing and 25 days in air

CC = 1 day in form and 27 days with curing compound

SC = 16 hours of steam curing

7-MC = 7 days moist curing

28-MC = 28 days moist curing

90-MC = 90 days moist curing

the comparisons of the effects of aggregate type for various curing conditions. It can be seen that the modulus of rupture of concretes using Calera aggregate is significantly higher than those using Brooksville or river gravel aggregate for all curing conditions, except for air curing (3 days moist curing and 25 days in air) and curing compound. The concretes using river gravel aggregate exhibited the same modulus of rupture as those using Brooksville aggregate for four curing conditions, namely air curing, curing compound, steam curing and 7-day moist curing. For the 28-day and 90-day moist curing, the concretes using Brooksville aggregate produced lower modulus of rupture than those using river gravel by about 11% and 17%, respectively.

Table 7.4 shows the comparison of the effects of cement type. The results of Duncan's test show that the modulus of rupture of concretes using cement Type III is significantly higher than those using cement Type I or II. Cement Type I produced the same modulus of rupture as cement Type II.

Table 7.5 displays the comparison of the effects of curing conditions for various aggregate types. Concretes that were cured in a moist room for 90 days produced the highest modulus of rupture for all aggregate types as compared with the other five curing conditions. The 28 days moist-cured specimens gave higher modulus of rupture than the air-cured and steam-cured specimens and the specimens with the curing compound. The modulus of rupture of the air-cured and curing compound specimens are not significantly different from each other for all aggregate types. The modulus of rupture of the steam-cured specimens is slightly lower than the 7-day moist-cured specimens, but they are not

Table 7.4 Grouping of Cement Type on the Basis of Modulus of Rupture from Duncan's Test (Design No. 1)

Cement Type	Means (psi)	Grouping
III	890	A
I	804	B
II	753	B

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

Table 7.5 Grouping of Curing Condition by Aggregate Type on the Basis of Modulus of Rupture from Duncan's Test (Design No. 1)

Aggregate Type	Curing Condition*	Means (psi)	Grouping
Brooksville	90-MC	818	A
	28-MC	807	A
	7-MC	744	A B
	SC	719	B
	AC	687	B
	CC	688	B
Calera	90-MC	1110	A
	28-MC	1002	B
	7-MC	971	B
	SC	889	C
	AC	756	D
	CC	732	D
River Gravel	90-MC	986	A
	28-MC	903	B
	7-MC	797	C
	SC	732	C D
	AC	675	D
	CC	673	D

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

significantly different at an α level of 0.05 for concretes using Brooksville and river gravel aggregates.

7.2.2 Results of Experimental Design No. 2

This section presents the results of experimental design no. 2, which dealt with the effects of aggregate type, w/c-ratio and curing condition on the modulus of rupture. The results of tests on 18 batches, as shown in Table C.1 in Appendix C.1, were used for analysis in a manner similar to that done for design no. 1. Mix parameters and batch numbers for all 18 batches are displayed in Table 6.3. Table 7.6 presents the mean moduli of rupture of concretes using three different aggregate types and three different w/c-ratios at the six curing conditions.

ANOVA was performed on the modulus of rupture data using the mathematical model as expressed by Equation 6.2. Results of ANOVA on the modulus of rupture data are summarized in Table 7.7. From Table 7.7, it can be seen that the effects of aggregate type (A), w/c-ratio (W), curing condition (D), and the interactions AD and WD are significant at an α level of 0.05 while the interactions AW and AWD are not significant. The significance of the effects of aggregate type, curing condition and the interaction AD are in agreement with the ANOVA results from design no. 1. Therefore, from the ANOVA results, it follows that:

1. Since aggregate type interacts with curing condition and does not interact with w/c-ratio, the effects of aggregate type can be evaluated by comparing the means of modulus of rupture, averaged over the w/c-ratios at each curing condition.
2. Since w/c-ratio interacts with curing condition and does not interact with aggregate type, the effects w/c-ratio can be

Table 7.6 Mean Moduli of Rupture of Concrete in Design No. 2

Aggregate Type	W/C-Ratio	Modulus of Rupture (psi)					
		Curing Condition*					
		1	2	3	4	5	6
Brooksville	0.45	617	631	688	554	626	688
	0.38	674	638	674	597	603	720
	0.33	682	779	821	680	665	809
Calera	0.45	713	758	782	659	643	923
	0.38	783	859	872	708	688	948
	0.33	837	939	967	797	750	1119
River Gravel	0.45	560	586	676	617	627	736
	0.38	641	710	769	638	638	848
	0.33	709	745	884	697	693	908

Note: * See footnotes on Table 7.1

Table 7.7 Result of ANOVA on Modulus of Rupture for Design No. 2

Factor	df	MS	F-Value	P
A	2	417439.7	57.18	0.0001*
W	2	322440.7	44.17	0.0001*
AW	4	6847.2	0.94	0.4845
B(AW)	9	7300.2	-	-
D	5	222676.3	88.26	0.0001*
AD	10	17436.1	6.91	0.0001*
WD	10	6075.1	2.41	0.0218*
AWD	20	2122.8	0.84	0.6541
BD(AW)	45	2522.9	-	-
ϵ	108	1356.2		

Note: * Significant at α level of 0.05

evaluated by comparing the means of modulus of rupture for three w/c-ratios, averaged over the aggregate types at each curing condition.

3. Since curing condition interacts with the other two factors, the effects caused by the six curing conditions can be assessed by comparing the means of modulus of rupture for six curing conditions, averaged at each aggregate type and w/c-ratio.

The Duncan's multiple-range test at an α level of 0.05 was used to compare the means of modulus of rupture. Table 7.8 shows the comparison of the effects of aggregate type for various curing conditions. It can be seen that the concretes using Calera aggregate gave higher modulus of rupture than those using Brooksville or river gravel aggregate for all curing conditions. The concretes using Brooksville aggregate exhibited the same modulus of rupture as those using river gravel for all curing conditions except for 90-day moist curing. At 90-day moist curing, the concrete using Brooksville aggregate had lower modulus of rupture than that using river gravel by about 11%.

Table 7.9 shows the comparison of the effects of w/c-ratio for various curing conditions. It can be seen that the modulus of rupture generally increases as the w/c-ratio decreases.

Table 7.10 shows the comparison of the effects of curing conditions for various combinations of w/c-ratio and aggregate type. It can be seen that the AC and CC curing conditions gave the same modulus of rupture, while the 7-MC and SC curing conditions also gave the same modulus of rupture, for all aggregate types and w/c-ratios. The 28-MC curing condition gave higher modulus of rupture than the AC, CC, an SC curing conditions for all aggregate types and w/c-ratios. For concretes

Table 7.8 Grouping of Aggregate Type by Curing Condition on the Basis of Modulus of Rupture from Duncan's Test (Design No. 2)

Curing Condition*	Aggregate Type	Means (psi)	Grouping
AC	Calera	721	A
	Brooksville	611	B
	River Gravel	651	B
CC	Calera	694	A
	Brooksville	631	B
	River Gravel	652	A B
SC	Calera	778	A
	Brooksville	658	B
	River Gravel	640	B
7-MC	Calera	852	A
	Brooksville	682	B
	River Gravel	680	B
28-MC	Calera	874	A
	Brooksville	728	B
	River Gravel	776	B
90-MC	Calera	997	A
	Brooksville	739	C
	River Gravel	831	B

Note: - Means in the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

Table 7.9 Grouping of W/C-Ratio by Curing Condition on the Basis of Modulus of Rupture from Duncan's Test (Design No. 2)

Curing Condition*	W/C-Ratio	Means (psi)	Grouping
AC	0.33	725	A
	0.38	648	B
	0.45	610	B
CC	0.33	703	A
	0.38	643	B
	0.45	632	B
SC	0.33	742	A
	0.38	700	A
	0.45	633	B
7-MC	0.33	821	A
	0.38	736	B
	0.45	658	C
28-MC	0.33	891	A
	0.38	772	B
	0.45	715	B
90-MC	0.33	945	A
	0.38	839	B
	0.45	782	B

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

Table 7.10 Grouping of Curing Condition by Aggregate Type and W/C-Ratio on the Basis of Modulus of Rupture from Duncan's Test (Design No. 2)

Aggregate Type	Curing Condition*	Duncan's Grouping for Each W/C-Ratio			
		0.33	0.38	0.45	
Brooksville	90-MC	A (809)	A (720)	A (688)	
	28-MC	A (821)	AB (674)	A (688)	
	AC	BC (680)	B (597)	B (554)	
	CC	C (665)	B (603)	AB (626)	
	7-MC	AB (779)	AB (638)	AB (631)	
	SC	BC (682)	AB (674)	AB (617)	
Calera	90-MC	A (1119)	A (948)	A (923)	
	28-MC	B (967)	AB (872)	B (782)	
	AC	D (797)	C (708)	CD (659)	
	CC	D (750)	C (688)	D (643)	
	7-MC	BC (939)	AB (859)	BC (758)	
	SC	CD (837)	BC (783)	BCD (713)	
River Gravel	90-MC	A (908)	A (848)	A (736)	
	28-MC	A (884)	AB (769)	AB (676)	
	AC	B (697)	C (638)	BC (617)	
	CC	B (693)	C (638)	BC (627)	
	7-MC	B (745)	BC (710)	C (586)	
	SC	B (709)	C (641)	C (560)	

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

** Means of modulus of rupture (psi)

using river gravel, the AC, CC, SC and 7-MC curing conditions gave the same modulus of rupture for all w/c-ratios.

7.2.3 Results of Experimental Design No. 3

The results of experimental design no. 3, which dealt with the effects of w/c-ratio, cement type and curing condition on the modulus of rupture are presented in this section. The results of tests on 18 batches, as shown in Table C.1 in Appendix C, were analyzed in a manner similar to that done for design no. 1. Mix parameters and batch numbers for all 18 batches are presented in Table 6.4. Comparison of mean modulus of rupture of concretes using three different w/c-ratios and three different cement types at the six curing conditions are presented in Table 7.11.

ANOVA was performed on the modulus of rupture data by using the linear model as shown by Equation 6.3. Results of ANOVA are summarized in Table 7.12. It can be seen from Table 7.12 that the effects of w/c-ratio (W), cement type (C), and curing condition (D) are significant while all the interactions are insignificant. Therefore, the comparison of means should be as follows:

1. Since w/c-ratio does not interact with the other two factors, the effects of w/c-ratio can be evaluated by comparing the means of modulus of rupture for three w/c-ratios, averaged over the other two factors.
2. Since cement type does not interact with the other two factors, the effects of cement can be evaluated by comparing the means of modulus of rupture for three cement types, averaged over the other two factors.

Table 7.11 Mean Moduli of Rupture of Concrete Design No. 3

W/C-Ratio	Cement Type	Modulus of Rupture (psi)					
		Curing Condition*					
		1	2	3	4	5	6
0.45	I	617	631	688	555	626	688
	II	623	628	633	602	620	698
	III	660	754	727	542	569	758
0.38	I	675	638	674	597	603	720
	II	681	701	687	510	559	725
	III	743	777	780	606	606	816
0.33	I	682	779	821	681	665	809
	II	654	630	735	629	630	779
	III	823	822	864	752	769	867

Note: * See footnotes on Table 7.1

Table 7.12 Result of ANOVA on Modulus of Rupture for Design No. 3

Factor	df	MS	F-Value	P
W	2	182585.8	16.11	0.0011*
C	2	140170.4	12.37	0.0026*
WC	4	17126.1	1.51	0.2782
B(WC)	9	11334.5	-	-
D	5	126891.4	42.05	0.0001*
WD	10	5345.1	1.77	0.0941
CD	10	4409.1	1.46	0.1858
WCD	20	4665.8	1.55	0.1123
BD(WC)	45	3017.8	-	-
ϵ	108	654.6		

Note: * Significant at α level of 0.05

3. Since curing condition does not interact with the other two factors, the effects caused by the six curing conditions can be assessed by comparing the means of modulus of rupture for the six curing conditions, averaged over the other two factors.

The comparison of means was done by Duncan's test at an α level of 0.05. The results are summarized in Tables 7.13 through 7.15.

Table 7.13 shows the comparison of the effects of w/c-ratio. It can be seen that the concrete with a w/c-ratio of 0.33 has a higher modulus of rupture than that with w/c-ratios of 0.38 and 0.45. The concrete with a w/c-ratio of 0.38 gave slightly higher modulus of rupture than that with a w/c-ratio of 0.45, but they are not significantly different from each other at an α level of 0.05.

Table 7.14 shows the comparison of the effects of cement type. The modulus of rupture of concrete using Type III cement is higher than those using Type I or II cement. Cement Types I and II produced about the same modulus of rupture.

From Table 7.15 which shows the comparison of the effects of curing conditions, it can be noted that the 90-day moist curing produced the highest modulus of rupture as compared with the other five curing conditions in this study. The 28-MC curing condition gave higher modulus of rupture than the AC and CC curing conditions. The modulus of rupture of air-cured and curing-compound specimens are not significantly different from each other. Also, the modulus of rupture of steam-cured specimens is not significantly different from that obtained from 7 days moist-cured specimens. The effects of the curing condition as observed from this design are in agreement with those observed in design nos. 1 and 2.

Table 7.13 Grouping of W/C-Ratio on the Basis of Modulus of Rupture from Duncan's Test (Design No.3)

W/C-Ratio	Means (psi)	Grouping
0.33	744	A
0.38	672	B
0.45	647	B

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

Table 7.14 Grouping of Cement Type on the Basis of Modulus of Rupture from Duncan's Test (Design No. 3)

Cement Type	Means (psi)	Grouping
III	737	A
I	675	B
II	651	B

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

Table 7.15 Grouping of Curing Condition on the Basis of Modulus of Rupture from Duncan's Test (Design No. 3)

Curing Condition*	Means (psi)	Grouping
90-MC	762	A
28-MC	734	B
7-MC	706	C
SC	684	C
CC	627	D
AC	611	D

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

7.2.4 Results of Experimental Design No. 4 (Fly Ash Concrete)

The results of experimental design no. 4, which dealt with the effects of w/c-ratio, fly ash and curing condition on the modulus of rupture are presented in this section. The results of tests on 12 batches, as shown in Table C.1 in Appendix C, were analyzed in a manner similar to the previous three designs. Mix parameters and batch numbers for all 12 batches are shown in Table 6.5. The mean moduli of rupture of the control concrete (without fly ash) and fly ash concrete (with 20% of fly ash replacement by weight of cement) are displayed in Table 7.16.

Figures 7.1 and 7.2 show plots of moduli of rupture as functions of water-cementitious ratio for the 28-day and 90-day moist curing, respectively. It can be seen that for these two curing conditions, the modulus of rupture generally increases as the water-cementitious ratio decreases. However for the air-curing, curing-compound, and steam-curing conditions, the modulus of rupture of the fly ash concrete does not change much as the water-cementitious ratio changes, as observed from Table 7.16.

ANOVA was performed on the test results by using the linear model shown in Equation 6.4. Results of ANOVA were summarized in Table 7.17. From Table 7.17, it can be seen that the significant factors are curing condition (D) and the three-way interaction of fly ash replacement, w/c-ratio and curing condition (FWD). The other factors are statistically insignificant. The insignificance of w/c-ratio (w) is not in agreement with the analysis results from design nos. 2 and 3.

Table 7.16 Mean Moduli of Rupture of Control Concrete and Fly Ash Concrete

Percent Fly Ash	W/(C+F) Ratio	Modulus of Rupture (psi)					
		Curing Condition*					
		1	2	3	4	5	6
0	0.45	623	628	633	602	620	698
0	0.38	681	701	687	510	559	725
0	0.33	654	630	735	628	629	779
20	0.45	546	581	625	571	564	705
20	0.38	543	582	643	554	579	753
20	0.33	610	707	712	513	514	738

Note: * See footnotes on Table 7.1

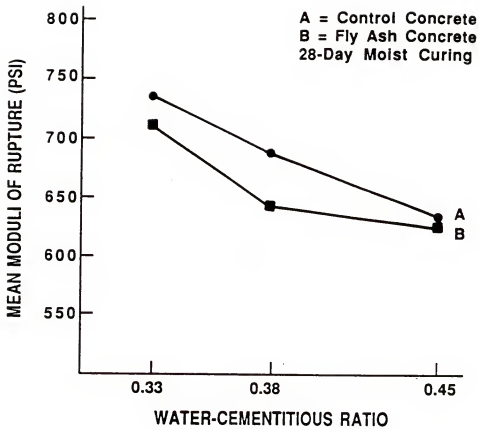


Figure 7.1 Effects of Water-Cementitious Ratio on the Modulus of Rupture of Control and Fly Ash Concretes at 28-Day Moist Curing

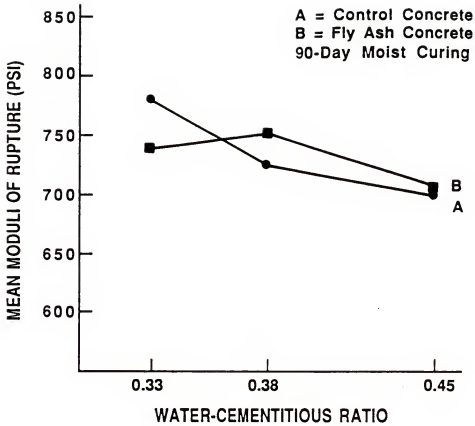


Figure 7.2 Effects of Water-Cementitious Ratio on the Modulus of Rupture of Control and Fly Ash Concretes at 90-Day Moist Curing

Table 7.17 Result of ANOVA on Modulus of Rupture for Design No. 4

Factor	df	MS	F-Value	P
F	1	51832.11	5.24	0.0621
W	2	18454.4	1.86	0.2346
FW	2	273.8	0.03	0.9728
B(FW)	6	9898.3	-	-
D	5	96499.3	23.36	0.0001*
FD	5	4827.3	1.17	0.3475
WD	10	5395.3	1.31	0.2718
FWD	10	9745.5	2.36	0.0339*
BD(FW)	30	4131.2	-	-
ϵ	72	864.65		

Note: * Significant at α level of 0.05

Since the three-factor interaction (FWD) is significant, none of the main factors is acting independently [54]. The comparison of means must be done as follows:

1. Comparison of w/c-ratio must be made at each percent of fly ash and curing condition.
2. Comparison of percent of fly ash must be made at each w/c-ratio and curing condition.
3. Comparison of curing condition must be made at each w/c-ratio and percent of fly ash.

The Duncan's multiple-range test at a level of significance of 0.05 was used to compare the means of modulus of rupture. The results are summarized in Tables 7.18 through 7.20.

Table 7.18 shows the comparison of the effects of w/c-ratio. It can be seen that the w/c-ratio has no significant effect on the modulus of rupture for both control and fly ash concretes and all curing conditions. For moist-cured specimens, the concrete with a w/c-ratio of 0.33 produced higher modulus of rupture than the concretes with the other two w/c-ratios, but they are not significantly different from each other at an α level of 0.05.

Table 7.19 shows the comparison of the effects of fly ash replacement. It shows that fly ash concrete gives the same modulus of rupture as the control concrete for all cases, except for the concretes with a w/c-ratio of 0.38 and SC curing condition.

Table 7.20 shows the comparison of the effects of curing conditions. It can be seen that the 90-day moist-curing condition gives the highest modulus of rupture, but it is not significantly different from the 28-MC curing condition for all cases. The AC and CC curing

Table 7.18 Grouping of W/C-Ratio by Percent of Fly Ash Replacement and Curing Condition on the Basis of Modulus of Rupture from Duncan's Test (Design No. 4)

Percent Fly Ash	W/C-Ratio	Duncan's Grouping for Each Curing Condition					
		AC*	CC	SC	7-MC	28-MC	90-MC
0	0.33	A (628)	A (629)	A (654)	A (630)	A (735)	A (779)
	0.38	A (510)	A (559)	A (681)	A (701)	A (687)	A (725)
	0.45	A (602)	A (620)	A (623)	A (628)	A (633)	A (698)
20	0.33	A (513)	A (514)	A (610)	A (707)	A (712)	A (738)
	0.38	A (554)	A (579)	A (543)	A (582)	A (643)	A (753)
	0.45	A (571)	A (564)	A (546)	A (581)	A (625)	A (705)

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Tables 7.3

** Mean of Modulus of rupture (psi)

Table 7.19 Grouping of Percent of Fly Ash Replacement by W/C-Ratio and Curing Condition on the Basis of Modulus of Rupture from Duncan's Test (Design No. 4)

W/C-Ratio	Percent Fly Ash	Duncan's Grouping for Each Curing Condition					
		AC*	CC	SC	7-MC	28-MC	90-MC
0.33	0	A (628)	A (629)	A (654)	A (630)	A (735)	A (779)
	20	A (513)	A (514)	A (610)	A (707)	A (712)	A (738)
0.38	0	A (510)	A (579)	A (681)	A (701)	A (687)	A (725)
	20	A (554)	A (559)	B (543)	A (582)	A (643)	A (753)
0.45	0	A (602)	A (620)	A (623)	A (628)	A (633)	A (698)
	20	A (571)	A (564)	A (546)	A (581)	A (625)	A (705)

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Tables 7.3

** Mean of Modulus of rupture (psi)

Table 7.20 Grouping of Curing Condition by W/C-Ratio and Percent of Fly Ash Replacement on the Basis of Modulus of Rupture from Duncan's Test (Design No. 4)

Percent Fly Ash	Curing Condition*	Duncan's Grouping for Each W/C-Ratio			
		0.33	0.38	0.45	
		**			
0	90-MC	A (779)	A (725)	A (698)	
	28-MC	A B (735)	A (687)	A (633)	
	7-MC	B (630)	A (701)	A (628)	
	SC	A B (654)	A (681)	A (623)	
	AC	B (628)	B (510)	A (602)	
	CC	B (629)	B (559)	A (620)	
20	90-MC	A (738)	A (753)	A (705)	
	28-MC	A (712)	A B (643)	A B (625)	
	7-MC	A (707)	B (582)	A B (581)	
	SC	A B (610)	B (543)	B (546)	
	AC	B (513)	B (554)	A B (571)	
	CC	B (514)	B (579)	A B (564)	

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

** Mean of modulus of rupture (psi)

conditions gave the same modulus of rupture, while the 7-MC and SC curing conditions also gave the same modulus of rupture for all cases. The 28-MC curing condition gave higher modulus of rupture than the AC and CC curing conditions for all cases. For the control concrete with w/c-ratio of 0.45, the curing condition has no significant effect on the modulus of rupture.

From the results of the comparison of means, it can be seen that the curing condition is the most influential in the significance of the the three-factor interaction (FWD).

7.2.5 Results of Experimental Design No. 5

The results of experimental design no. 5, which dealt with the effects of maximum aggregate size and curing condition on the modulus of rupture are presented in this section. The results of tests on six batches, as shown in Table C.1 in Appendix C, were used in the analysis for this design. Mix parameters and batch numbers for all these six batches are presented in Table 6.6. Table 7.21 presents mean moduli of rupture of concretes using three different maximum aggregate sizes at the six curing conditions.

ANOVA was performed on the modulus of rupture data by using the linear model shown in Equation 6.5. Results of ANOVA are summarized in Table 7.22. From Table 7.22, it can be seen that the effect of curing condition (D) is significant while the effects of maximum aggregate size (M) and the interaction MD are insignificant. It follows that:

1. Since maximum aggregate size does not interact with the curing condition, the effect of maximum aggregate sizes can be evaluated by comparing the means of modulus of rupture for maximum aggregate sizes, averaged over the curing condition. Although

Table 7.21 Mean Moduli of Rupture of Concrete in Design No. 5

Maximum Aggregate Size (inches)	Modulus of Rupture (psi)					
	Curing Condition*					
	1	2	3	4	5	6
3/8	684	749	794	649	667	782
3/4	682	779	821	681	665	809
1	635	708	755	625	629	772

Note: * See footnotes on Table 7.1

Table 7.22 Result of ANOVA on Modulus of Rupture for Design No. 5

Factor	df	MS	F-Value	P
M	2	16696.5	5.72	0.0947
B(M)	3	2920.0	-	-
D	5	52695.6	59.67	0.0001*
MD	10	438.4	0.50	0.8845
ϵ	51	883.2		

Note: * Significant at α level of 0.05

the effect of maximum aggregate size is insignificant, the comparison means is done to further clarify the results of the ANOVA.

2. Since curing condition does not interact with the maximum aggregate size, the effects caused by the six curing conditions can be assessed by comparing the means of modulus of rupture for six curing conditions, averaged over the other factor.

The Duncan's multiple-range test at a level of significance of 0.05 was used to compare the means of modulus of rupture. The results are summarized in Tables 7.23 and 7.24.

Table 7.23 shows the comparison of the effects of maximum aggregate size. It can be seen that the effect of maximum aggregate size on modulus of rupture is statistically not significant.

Table 7.24 shows the comparison of the effects of curing condition. It shows that the 90-day and 28-day moist-curing conditions were not significantly different from one another, while they produced significantly higher modulus of rupture than the other four curing conditions. The 7-day moist-curing condition gave slightly higher modulus of rupture than the SC curing condition. The AC, CC, and SC curing conditions gave about the same modulus of rupture.

7.2.6 Results of Experimental Design No. 6 (Silica Fume Concrete)

This section presents the results of experimental design no. 6, which dealt with the effects of silica fume and curing condition on the modulus of rupture. The results of tests on two batches of control concrete shown in Table C.1 and eight batches of silica fume concrete shown in Tables C.10 and C.11 in Appendix C were analyzed in a manner

Table 7.23 Grouping of Maximum Aggregate Size on the Basis of Modulus of Rupture from Duncan's Test (Design No. 5)

Maximum Aggregate Size (inches)	Means (psi)	Grouping
$\frac{3}{4}$	739	A
$\frac{3}{8}$	720	A
1	687	A

Note: - Means with the same letter are not significantly different
($\alpha = 0.05$)

Table 7.24 Grouping of Curing Condition on the Basis of Modulus of Rupture from Duncan's Test (Design No. 5)

Curing Condition*	Means (psi)	Grouping
28-MC	790	A
90-MC	788	A
7-MC	745	B
SC	666	C
CC	653	C
AC	652	C

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

similar to the one for design no. 5. Mix parameters and batch numbers for all ten batches are presented in Table 6.8. Figure 7.3 shows plots of the mean moduli of rupture as functions of percent addition of silica fume for two curing conditions.

ANOVA was performed on the modulus of rupture data by using the mathematical model as shown in Equation 6.6. Results of ANOVA are summarized in Table 7.25. From Table 7.25, it is seen that the effects of curing condition (D) and the interaction of silica fume addition and curing condition (SD) are significant while the effect of percent addition of silica fume (S) is not significant. The significance of the interaction SD can be seen from Figure 7.3. For a 28-day moist curing, the modulus of rupture increases as the added silica fume increases up to 10%, then levels off beyond 10% addition. For 90-day moist curing, the modulus of rupture increases as the added silica fume increases up to 20% addition. The modulus of rupture at any percent addition of silica fume depends on the curing condition. Therefore, the comparison of means must be made as follows:

1. Since percent addition of silica fume interacts with the curing condition, the effects of percent addition of silica fume can be evaluated by comparing the means of modulus of rupture for five levels of percent addition of silica fume, averaged at each curing condition.
2. Since curing condition interacts with the percent addition of silica fume, the effects caused by the two curing conditions can be assessed by comparing the means of modulus of rupture for two curing conditions, averaged at each percent addition of silica fume.

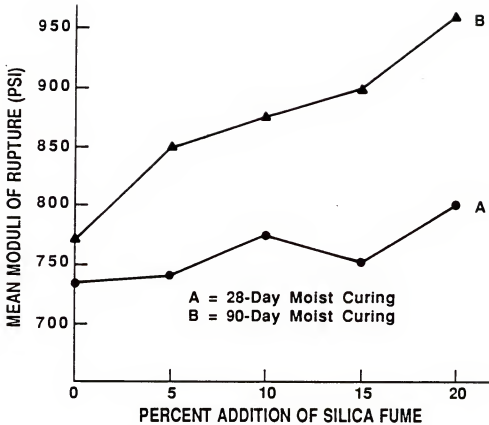


Figure 7.3 Effects of Silica Fume Addition on the Modulus of Rupture of Concrete with a Water-Cementitious Ratio of 0.33 and Brooksville Aggregate

Table 7.25 Result of ANOVA on Modulus of Rupture for Design No. 6

Factor	df	MS	F-Value	P
S	4	16620.2	2.72	0.1512
B(S)	5	6118.0	-	-
D	1	124880.6	87.74	0.0001*
SD	4	3996.6	2.81	0.0472*
ϵ	25	1423.3		

Note: * Significant at α level of 0.05

The Duncan's multiple-range test at a level of significance of 0.05 was used to compare the means of modulus of rupture. Table 7.26 shows the comparison of the effects of percent addition of silica fume. It can be seen from Table 7.26 that the 20% addition of silica fume produced higher modulus of rupture than the other 4% additions of silica fume for both curing conditions, but the differences between the 20%, 15% and 10% additions are not significant. The control concrete (without silica fume) gave about the same modulus of rupture as that with 5% addition of silica fume for both curing conditions. For the 28-day moist curing, the percent addition of silica fume had no effect on the modulus of rupture.

Table 7.27 shows the comparison of the effects of curing conditions. It shows that the 90-day moist-cured specimens produced higher modulus of rupture than the 28 days moist-cured specimens for all concretes with silica fume. For control concrete, the 90-MC and 28-MC curing conditions gave about the same modulus of rupture.

7.2.7 Results of Silica Fume Concrete Made with Calera Aggregate

This section presents the test results of silica fume concrete made with Calera aggregate. The results of tests on two batches of silica fume concrete are presented in Tables C.10 and C.11 in Appendix C. These results are compared with the results of tests on two batches of control concrete, which were made with Calera aggregate, cement Type II and a w/c-ratio of 0.33, and presented in Table C.1 in Appendix C (batch nos. 13 and 40).

Figure 7.4 shows plots of the mean moduli of rupture as functions of percent addition of silica fume for 28-day and 90-day moist curing.

Table 7.26 Grouping of Percent Addition of Silica Fume by Curing Condition on the Basis of Modulus of Rupture from Duncan's Test (Design No. 6)

Curing Condition*	Percent Addition of Silica Fume	Means (psi)	Grouping
28-MC	20	802	A
	10	775	A
	15	753	A
	5	741	A
	0	735	A
90-MC	20	960	A
	15	899	A B
	10	877	A B
	5	849	B C
	0	779	C

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

Table 7.27 Grouping of Curing Condition by Percent Addition of Silica Fume on the Basis of Modulus of Rupture from Duncan's Test (Design No. 6)

Percent Addition of Silica Fume	Curing Condition*	Means (psi)	Grouping
0	90-MC	779	A
	28-MC	735	A
5	90-MC	849	A
	28-MC	741	B
10	90-MC	877	A
	28-MC	775	B
15	90-MC	899	A
	28-MC	753	B
20	90-MC	960	A
	28-MC	802	B

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

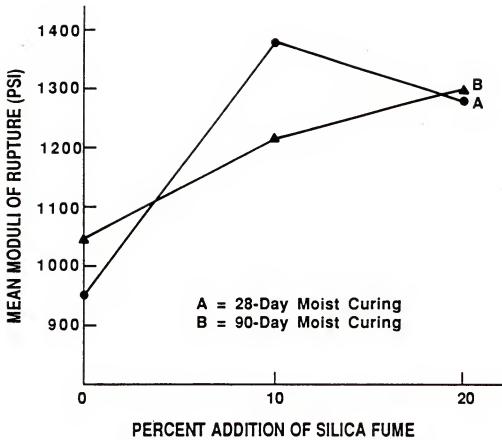


Figure 7.4 Effects of Silica Fume Addition on the Modulus of Rupture of Concrete with a Water-Cementitious Ratio of 0.33 and Calera Aggregate

From Figure 7.4, it can be seen that the silica fume concrete produced higher modulus of rupture than the control concrete. For 28-day moist curing, the 10% addition of silica fume gave the highest modulus of rupture. For 90-day moist curing, the modulus of rupture increases as the added silica fume increases. For the 10% addition, the 28-day moist curing gave a higher modulus of rupture than the 90-day moist curing. This discrepancy is believed to be caused by the large variability of the modulus of rupture test results.

7.3 Discussion of the Results

The results of a laboratory study on the modulus of rupture have been presented in the previous sections. In this section, these results are discussed in accordance with the objectives of this study.

7.3.1 Effect of Aggregate Type

According to the ANOVA results of design nos. 1 and 2, the effect of aggregate type on the modulus of rupture is significant at an α level of 0.05 (the probability of error of this conclusion is 0.05). The concrete using Calera aggregate gave higher modulus of rupture than those using river gravel and Brooksville aggregates. The Calera aggregate has a rough texture and angular shape. Therefore, the bond between the aggregate and hydrated cement paste is higher than that of a smooth and round aggregate (river gravel). Also, the high strength of the Calera aggregate is a factor for the higher modulus of rupture as compared with that of the Brooksville aggregate. Jones and Kaplan [38] have found that the surface texture and shape of coarse aggregate are the main factors of influence on the modulus of rupture, because these two properties of coarse aggregate affect the aggregate-cement bond. This agrees

with the test results in this study. Wright [5] also found results similar to this study.

For 28-day and 90-day moist curing, concretes using Brooksville aggregate gave lower modulus of rupture than concretes using river gravel by about 10% to 17%, but they gave about the same results for the other four curing conditions. The improvement of bond between the aggregate and cement paste due to the continued moist curing and the high strength of the river gravel are believed to be the main factors for this difference.

7.3.2 Effect of Cement Type

The type of cement used also affects the modulus of rupture. Concrete using cement Type III exhibited higher strength than concretes using cement Types I and II. Since Type III cement is a high early strength cement, it gains strength more rapidly than the other two cement types for up to about 1 month. In this study, five of the six curing conditions are at the age 28 days or less. Therefore, the strength of concretes using cement Type III should be higher than concretes using cements Types I and II for these five curing conditions. For 90-day moist curing, the test results of this study also showed higher modulus of rupture for concrete using cement Type III. Concrete using cement Type I produced higher modulus of rupture than those using cement Type II by about 5%, but they are not significantly different from each other at an α level of 0.05.

7.3.3 Effect of W/C-Ratio

Test results from design nos. 2 and 3 show that the w/c-ratio is one of the major parameters affecting the modulus of rupture. The higher the w/c-ratio, the lower the modulus of rupture would be.

For the fly ash concrete (in design no. 4), the effects of w/c-ratio do not follow the general trend for the conditions of air curing, curing compound and steam curing. At these conditions, the fly ash concrete with water-cementitious ratio of 0.45 gave the highest modulus of rupture and the water-cementitious ratio of 0.33 gave the lowest modulus of rupture. The shrinkage at the tension side of the tested specimens at those curing conditions may be the main factor that caused the variation in the results of modulus of rupture tests. For the moist-cured specimens, the specimens with lower w/c-ratio gave higher modulus than specimens with higher w/c-ratio, which is in agreement with the design nos. 2 and 3. The moist-cured specimens were tested within a short period of time after they were removed from the moist room. Therefore, shrinkage would have very little effect on the modulus of rupture. The reasons mentioned above are believed to be why the effect of w/c-ratio in design no. 4 does not agree with design nos. 2 and 3.

7.3.4 Effect of Maximum Aggregate Size

The maximum aggregate size has no significant effect on the modulus of rupture. The three maximum aggregate sizes, namely $\frac{3}{8}$ inch, $\frac{3}{4}$ inch and 1 inch were used in this study. These three sizes are in a very narrow range. The use of 1 inch maximum aggregate size gave the lowest modulus of rupture, as compared with the other two maximum aggregate sizes, but they were not significantly different from each other. The use of larger maximum aggregate size reduces the surface area of the aggregate, therefore the bond between aggregate and cement paste is less. This is believed to be the main factor contributing to why the 1 inch maximum aggregate size produced the lowest modulus of rupture in this study.

7.3.5 Effect of Fly Ash

The replacement of 20% of cement by fly ash has no significant effect on the modulus of rupture. From Table 7.16 and Figures 7.1 and 7.2, it can be seen that the control concrete produced slightly higher modulus of rupture than the fly ash concrete for the curing conditions of air curing, curing compound, steam curing, 7-day moist curing, and 28-day moist curing at all water-cementitious ratios. For 90-day moist curing, the fly ash concrete with water-cementitious ratios of 0.38 and 0.45 exhibited slightly higher modulus of rupture than the control concrete with the same w/c-ratio. However for the water-cementitious ratio of 0.33, the fly ash concrete gave a slightly lower modulus of rupture than the control concrete. Concrete with Class F fly ash which was used in this study has a lower rate of hydration than the control concrete, therefore, the early strength gain can be lower than that of the control concrete. At later ages, the strengths of both concretes are expected to be approximately the same [31,44]. These are in agreement with the test results from this study.

7.3.6 Effect of Silica Fume

7.3.6.1 Silica fume concrete made with Brooksville aggregate. The addition of silica fume to control concrete by keeping the same water-cementitious ratio tends to increase the modulus of rupture. The higher the percent addition of silica fume, the higher the modulus of rupture would be. This result can be clearly seen in Figure 7.3. The 20% addition of silica fume gave the highest modulus of rupture for both the 28-day and 90-day moist-curing conditions. The silica fume additions increase the modulus of rupture of the control concrete by 10% to 25% at 90-day moist curing, and only by 1% to 10% at 28-day moist curing.

Therefore, for 90 days moist-cured specimens, the addition of silica fume results in a higher efficiency of improvement of modulus of rupture than specimens with 28-day moist curing.

Silica fume is a pozzolanic material. It contributes to concrete strength in a manner very similar to fly ash and slag, but it is a more efficient pozzolan than fly ash because the pozzolanic reactions are evident at much earlier ages as compared with those of fly ash [21]. For continued moist curing, silica fume still continues to react with the products from hydrated cement and reduces the volume of large pores and capillaries in the cement pastes [28].

Jones and Kaplan [38] found that the upper limit of the modulus of rupture is set by the modulus of rupture of the mortar; the presence of coarse aggregate generally reduces the modulus of rupture of concrete below that of the mortar. The bond strength between the coarse aggregate and the mortar should be the main factor to indicate the magnitude of modulus of rupture. As the amount of addition of silica fume is increased, the bond strength between the coarse aggregate and the mortar will also be increased while the volume of pores and capillaries in the cement pastes is decreased. The reasons mentioned above can be used to explain why the 20% addition of silica fume exhibits the highest modulus of rupture for both curing conditions.

7.3.6.2 Silica fume concrete made with Calera aggregate. The modulus of rupture of silica fume concrete using Calera aggregate also exhibited higher than the control concrete at 28-day and 90-day moist-curing conditions. At 90-day moist curing, the 20% addition of silica fume gave the highest modulus of rupture, which agrees with the test results of silica fume concrete made with Brooksville aggregate. At

28-day moist curing, the 10% addition of silica fume gave the highest modulus of rupture. Since only two specimens for each curing condition were tested in this case, it is very difficult to draw any definite conclusions from this set of test results. A more extensive study is needed to investigate the modulus of rupture of silica fume concrete made with Calera aggregate.

7.3.7 Effect of Curing Condition

From the ANOVA results for all designs in this study, it can be concluded that the curing condition is one of the main significant factors affecting the modulus of rupture. The continued moist curing tends to increase the modulus of rupture. This is because the continued hydration of cement particles can reduce the pores in the cement paste and also improve the bond between the aggregate and the hydrated cement. The 90-day moist-curing condition produced the highest modulus of rupture. The 28-day moist-cured specimens exhibited higher modulus of rupture than the air-cured and steam-cured specimens and specimens cured with curing compound for all cases. Neilson [2] found that the modulus of rupture of beams cured in water for 21 days with additional air curing for 7 days has a tendency to decrease. It can be explained by the fact that, when air curing begins, the top surface dries out and tensile stresses are set up in the surface resulting in reduced modulus of rupture. In this study, the specimens were cured in a moist room for three days, followed by 25-day air curing. Since air-cured specimens in this study were cured in room environment for a longer period of time than in the Neilson study, the tensile stresses at the surface of the specimens can cause higher effects on the modulus of rupture. Curing compound can be used to retard or reduce evaporation of moisture from

concrete [30]. Again, the evaporation of moisture from the surface of specimens with curing compound can cause a reduction in the modulus of rupture for the same reason given for the air-cured specimens.

The modulus of rupture of the AC and CC curing conditions are not significantly different from each other for all the cases. It can be explained that this is due to the fact that the degrees of hydration for both curing conditions are almost the same.

The 7-day moist-cured and steam-cured specimens gave about the same modulus of rupture from all designs, except for design no. 1, in which the concrete using Calera aggregate at 7-MC curing condition exhibited a higher modulus of rupture than that at the SC curing condition by about 8%.

For fly ash concrete, the moduli of rupture of the steam-cured concretes are 87%, 85% and 86% of those of the 28-day moist-cured specimens for water-cementitious ratios of 0.45, 0.38 and 0.33, respectively. Therefore, the effectiveness of steam curing on modulus of rupture of fly ash concrete are about the same for all three water-cementitious ratios.

7.4 Summary of Results

The findings presented in this chapter on the effects of mix parameters and curing condition on the modulus of rupture are summarized as follows:

1. The w/c-ratio is the major parameter affecting the modulus of rupture. Generally, the lower the w/c-ratio, the higher the modulus of rupture would be.

2. Curing condition is also one of the major parameters affecting the modulus of rupture. The continued moist curing tends to increase the modulus of rupture. The 90-day moist-curing condition gives the highest modulus of rupture. The 28-MC curing condition shows higher modulus of rupture than the AC, CC, and SC curing conditions. The modulus of rupture of the AC and CC curing conditions are not significantly different from each other. The 7-MC curing condition generally gives about the same modulus of rupture as the SC curing condition.
3. The aggregate type has some effects on the modulus of rupture. Concretes using Calera aggregate show higher modulus of rupture than concretes using Brooksville and river gravel aggregates. Concretes using Brooksville and river gravel aggregates exhibit about the same modulus of rupture for the curing conditions of AC, CC, SC and 7-MC. For 28-day and 90-day moist curing, the moduli of rupture of concretes using Brooksville aggregate are lower than those using river gravel aggregate by about 10% to 17%.
4. Cement type has some effect on the modulus of rupture. Cement Type III gives higher modulus of rupture than cement Types I and II. Concretes using cement Type I show about the same modulus of rupture of concretes using cement Type II.
5. Maximum aggregate sizes of $\frac{3}{8}$ inch, $\frac{3}{4}$ inch and 1 inch show the same modulus of rupture (are not significantly different from one to another at an α level of 0.05).
6. The modulus of rupture of fly ash concrete is lower than the control concrete for the AC, CC, SC and 7-MC curing conditions,

but they are not different from each other at an α level of 0.05. For 28-day and 90-day moist-curing conditions, the modulus of rupture of fly ash concrete shows about the same as the control concrete. The ANOVA results show that the replacement of 20% of cement by fly ash has no effect on the modulus of rupture.

7. The silica fume concrete made with Brooksville aggregate produces the maximum modulus of rupture as a silica fume addition of approximately 20% by weight of cement.
8. The silica fume concrete made with Calera aggregate also produces the maximum modulus of rupture at 20% addition of silica fume for the 90-day moist-cured specimens.

CHAPTER 8 RESULTS OF COMPRESSIVE STRENGTH TESTS

8.1 Introduction

Compressive strength is one of the properties of concrete that is commonly used to control the quality of concrete. It can also be used as a guide for the control of the durability of the concrete structure. Because of its importance and significance as a major indicator of concrete performance, the compressive strength was included in this research program.

This chapter presents the results of compressive strength tests for all six experimental designs. The details of all experimental designs are presented in Chapter 6. The general laboratory procedure described in Chapter 4 was used to prepare and test the cylindrical specimens. The method of analysis on the obtained data are also described in Chapter 6.

8.2 Analysis of Results

8.2.1 Results of Experimental Design No. 1

The results of experimental design no. 1, which dealt with the effects of aggregate type, cement type and curing condition on the compressive strength are presented in this section. The compressive strength test results, which are shown in Table C.2 in Appendix C, were analyzed with the aid of the ANOVA statistical method. Mix parameters and batch numbers for all 18 batches are displayed in Table 6.1. Table

8.1 presents the mean compressive strengths of the concretes in design no. 1 using three different aggregates and cement types at the six curing conditions.

ANOVA was performed on the compressive strength data by using the linear model as shown by Equation 6.1. Results of ANOVA are summarized in Table 8.2. From Table 8.2, it can be seen that the effects of cement type (C) and curing condition (D) and the interactions AD and CD are significant while the effects of aggregate type (A) and the interactions AC and ACD are insignificant. It, therefore, follows that:

1. Since aggregate type interacts with curing condition (and not cement type), comparison of the effects of aggregate types must be made separately for each curing condition, averaged over the three cement types.
2. Since cement type interacts with curing condition (and not aggregate type), comparison of the effects of cement types must be made separately for each curing condition, averaged over the three aggregate types.
3. Since curing condition interacts with aggregate and cement types, comparison of the effects of curing conditions can be made separately for each aggregate and cement types.

The Duncan's multiple-range test at an α level (probability of error) of 0.05 was used to compare the means of compressive strength. Table 8.3 shows the comparisons of the effects of aggregate type for various curing conditions. It can be seen that for all curing conditions except the curing compound condition, the concrete mixes using Calera aggregate gave significantly higher compressive strength than those using Brooksville or river gravel aggregate, while those using

Table 8.1 Mean Compressive Strengths of Concrete in Design No. 1

Aggregate Type	Cement Type	Compressive Strength (psi)					
		Curing Condition*					
		1	2	3	4	5	6
Brooksville	I	6744	7117	8933	8760	8797	9440
	II	6533	6656	8459	8058	8152	9081
	III	8501	8376	10085	9805	9787	10182
Calera	I	7633	8081	9939	9566	9387	11117
	II	7498	7378	9186	8492	8127	10433
	III	8863	8942	10126	9888	9472	10828
River Gravel	I	7257	7063	8586	8627	8386	9413
	II	6365	6330	8629	8454	7710	9550
	III	8341	8439	9895	9736	9752	10393

Note: * Curing conditions:

1 = 16 hours steam curing

2 = 7 days moist curing

3 = 28 days moist curing

4 = 3 days moist curing and 25 days in air

5 = 1 day in form and 27 days with curing compound

6 = 90 days moist curing

Table 8.2 Result of ANOVA on Compressive Strength for Design No. 1

Factor	df	MS	F-Value	P
A	2	10259047.5	3.15	0.0917
C	2	39343459.2	12.09	0.0028*
AC	4	1241602.1	0.38	0.8166
B(AC)	9	3254678.1	-	-
D	5	35442473.1	329.89	0.0001*
AD	10	480320.3	4.47	0.0002*
CD	10	575106.1	5.35	0.0001*
ACD	20	169169.44	1.57	0.1031
BD(AC)	45	107438.4	-	-
ϵ	108	42202.2		

Note: * Significant at α level of 0.05

Table 8.3 Grouping of Aggregate Type by Curing Condition on the Basis of Compressive Strength from Duncan's Test (Design No. 1)

Curing Condition*	Aggregate Type	Means (psi)	Grouping
AC	Calera	9315	A
	Brooksville	8875	B
	River Gravel	8939	A B
CC	Calera	8995	A
	Brooksville	8912	A
	River Gravel	8616	A
SC	Calera	7998	A
	Brooksville	7259	B
	River Gravel	7321	B
7-MC	Calera	8134	A
	Brooksville	7383	B
	River Gravel	7277	B
28-MC	Calera	9750	A
	Brooksville	9159	B
	River Gravel	9037	B
90-MC	Calera	10792	A
	Brooksville	9568	B
	River Gravel	9785	B

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

Brooksville aggregate exhibited the same compressive strength as those using river gravel aggregate. For the CC curing condition, all three aggregates produced the same compressive strength.

Table 8.4 presents the comparison of the effects of cement type for various curing conditions. The concretes using Type III cement had significantly higher compressive strengths than those using Type I or Type II cement for all six curing conditions. Type I cement produced significantly higher compressive strength than Type II cement for all curing conditions except the 90-day moist-curing condition, at which they produced approximately equal compressive strengths.

Table 8.5 shows the comparison of the effects of curing conditions for various aggregate and cement types. It can be seen that the 90-day moist-curing condition resulted in the highest compressive strength of all curing conditions. The AC and CC curing conditions resulted in approximately the same compressive strength, but the compressive strength produced by these two curing conditions were slightly lower than those by the 28-day moist-curing condition. The SC and 7-MC curing conditions gave approximately the same compressive strength, but they were both lower than those from the other four curing conditions for all aggregate and cement types. It can also be seen that the difference between the 28-MC and 90-MC is less significant for the Brooksville aggregate than for the Calera and river gravel aggregates. The difference between the 28-MC and 90-MC is also less significant for the Type III cement than for the Type I or Type II cement.

8.2.2 Results of Experimental Design No. 2

This section presents the results of experimental design no. 2, which dealt with the effects of aggregate type, w/c-ratio and curing

Table 8.4 Grouping of Cement Type by Curing Condition on the Basis of Compressive Strength from Duncan's Test (Design No. 1)

Curing Condition*	Cement Type	Means (psi)	Grouping
AC	III	9810	A
	I	8984	B
	II	8335	C
CC	III	9670	A
	I	8857	B
	II	7996	C
SC	III	8568	A
	I	7211	B
	II	6798	C
7-MC	III	8585	A
	I	7420	B
	II	6788	C
28-MC	III	10035	A
	I	9152	B
	II	8758	C
90-MC	III	10468	A
	I	9990	B
	II	9688	B

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

Table 8.5 Grouping of Curing Condition by Aggregate Type and Cement Type on the Basis of Compressive Strength from Duncan's Test (Design No. 1)

Aggregate Type	Curing Condition*	Duncan's Grouping for Each Cement Type					
		I		II		III	
Brooksville	90-MC	A	** (9440)	A	(9081)	A	(10182)
	28-MC	A	(8933)	AB	(8459)	A	(10085)
	AC	A	(8760)	B	(8058)	A	(9805)
	CC	A	(8797)	B	(8152)	A	(9787)
	7-MC	B	(7117)	C	(6656)	B	(8376)
	SC	B	(6744)	C	(6533)	B	(8501)
Calera	90-MC	A	(11117)	A	(10433)	A	(10828)
	28-MC	B	(9939)	B	(9186)	B	(10126)
	AC	B	(9566)	C	(8492)	B	(9888)
	CC	B	(9387)	C	(8127)	BC	(9472)
	7-MC	C	(8081)	D	(7378)	C	(8942)
	SC	C	(7633)	D	(7498)	C	(8863)
River Gravel	90-MC	A	(9413)	A	(9550)	A	(10393)
	28-MC	B	(8586)	B	(8629)	A	(9895)
	AC	B	(8627)	B	(8454)	A	(9736)
	CC	B	(8386)	C	(7710)	A	(9752)
	7-MC	C	(7063)	D	(6330)	B	(8439)
	SC	C	(7257)	D	(6365)	B	(8341)

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

** Mean of compressive strength (psi)

condition on the compressive strength. The results of tests on 18 batches from Table C.2 in Appendix C were used for analysis in a manner similar to that done for the design no. 1. Mix parameters and batch numbers for all 18 batches are presented in Table 6.3. The mean compressive strengths of concretes in this design using three different aggregate types, three levels of w/c-ratio and six curing conditions are presented in Table 8.6.

ANOVA was performed on the compressive strength data by using the linear model as shown by Equation 6.2. Results of ANOVA on the compressive strength are summarized in Table 8.7. From Table 8.7, it is seen that the effects of aggregate type (A), w/c-ratio (W), curing condition (D) and the interaction AD are significant while the effects of interactions AW, WD and AWD are not significant. Therefore, the comparison of means for this design must be made as follows:

1. Since aggregate type interacts with curing condition (and not w/c-ratio), comparison of the effects of aggregate types must be made separately for each curing condition, averaged over the three w/c-ratios.
2. Since w/c-ratio does not interact with either aggregate type and curing condition, comparison of the effects of w/c-ratios can be made by averaging over all aggregate types and over all curing conditions.
3. Since curing condition interacts with aggregate type (and not w/c-ratio), comparison of the effects of curing condition must be made separately for each aggregate type, averaged over all w/c-ratios.

Table 8.6 Mean Compressive Strengths of Concrete in Design No. 2

Aggregate Type	W/C-Ratio	Compressive Strength (psi)					
		Curing Condition*					
		1	2	3	4	5	6
Brooksville	0.45	5158	5663	7503	7256	7372	8028
	0.38	5649	6287	7589	7704	7640	8324
	0.33	6744	7117	8933	8760	8797	9440
Calera	0.45	6098	6369	8306	7848	7373	8847
	0.38	6233	6721	8827	8416	8282	10103
	0.33	7633	8081	9939	9566	9387	11117
River Gravel	0.45	5155	5214	7139	6804	6327	7716
	0.38	6257	6392	8197	7929	7856	8318
	0.33	7257	7063	8586	8627	8386	9413

Note: * See footnotes on Table 8.1

Table 8.7 Result of ANOVA on Compressive Strength for Design No. 2

Factor	df	MS	F-Value	P
A	2	18696963.8	18.49	0.0006*
W	2	52836266.2	52.25	0.0001*
AW	4	954236.9	0.94	0.4817
B(AW)	9	1011153.5	-	-
D	5	42165501.9	448.52	0.0001*
AD	10	634784.8	6.75	0.0001*
WD	10	75304.2	0.80	0.6283
AWD	20	145224.7	1.54	0.1127
BD(AW)	45	94009.4	-	-
e	108	29988.32		

Note: * Significant at α level of 0.05

The Duncan's multiple-range test at an α level of 0.05 was used to compare the means of compressive strength. Table 8.8 shows the comparison of the effects of aggregate types for various curing conditions. It can be seen that the concretes made with Calera aggregate produced significantly higher compressive strength than those made with Brooksville or river gravel aggregate for all curing conditions. The compressive strength of the concrete using Brooksville and river gravel aggregates are not significantly different from each other for four curing conditions, namely AC, 7-MC, 28-MC and 90-MC. These results are in agreement with the results from design no. 1. For the CC curing condition, concretes using Brooksville aggregate exhibited higher compressive strength than concretes using river gravel aggregate by about 5%, but for SC curing condition, concretes using river gravel aggregate exhibited higher compressive strength by about 6%.

Table 8.9 displays the comparison of the mean effects of w/c-ratios. It is clear that the compressive strength increases as the w/c-ratio increases. The difference of means of compressive strength between one w/c-ratio and the other w/c-ratio is significant.

Table 8.10 shows the comparison of the effects of curing conditions for various aggregate types. It can be seen that the 90-MC curing condition gave the highest compressive strength for all aggregate types. The AC and CC curing conditions produced approximately the same compressive strength, but they gave slightly lower strength than the 28-MC curing condition for all aggregate types. The 7-day moist-cured and steam-cured specimens gave significantly lower compressive strength than the other four curing conditions. The 7-MC and SC curing conditions produced the same compressive strength, for the concretes using river

Table 8.8 Grouping of Aggregate Type by Curing Condition on the Basis of Compressive Strength from Duncan's Test (Design No. 2)

Curing Condition*	Aggregate Type	Means (psi)	Grouping
AC	Calera	8610	A
	Brooksville	7906	B
	River Gravel	7787	B
CC	Calera	8347	A
	Brooksville	7937	B
	River Gravel	7523	C
SC	Calera	6654	A
	Brooksville	5850	C
	River Gravel	6223	B
7-MC	Calera	7057	A
	Brooksville	6355	B
	River Gravel	6223	B
28-MC	Calera	9024	A
	Brooksville	8008	B
	River Gravel	7974	B
90-MC	Calera	10022	A
	Brooksville	8597	B
	River Gravel	8481	B

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

Table 8.9 Grouping of W/C-Ratio on the Basis of Compressive Strength from Duncan's Test (Design No. 2)

W/C-Ratio	Means (psi)	Grouping
0.33	8602	A
0.38	7595	B
0.45	6894	C

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

Table 8.10 Grouping of Curing Condition by Aggregate Type on the Basis of Compressive Strength from Duncan's Test (Design No. 2)

Aggregate Type*	Curing Condition*	Means (psi)	Grouping
Brooksville	90-MC	8597	A
	28-MC	8008	B
	AC	7906	B
	CC	7937	B
	7-MC	6355	C
	SC	5850	D
Calera	90-MC	10022	A
	28-MC	9024	B
	AC	8610	C
	CC	8347	C
	7-MC	7057	D
	SC	6654	E
River Gravel	90-MC	8481	A
	28-MC	7974	B
	AC	7787	B C
	CC	7523	C
	7-MC	6223	D
	SC	6223	D

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

gravel aggregate. However, for the concretes using Brooksville and Calera aggregates, the 7-MC curing condition produced higher compressive strength than the SC curing condition by about 8% and 6%, respectively. The effects caused by the six curing conditions in this design are in agreement with the design no. 1 for the majority of the cases.

8.2.3 Results of Experimental Design No. 3

This section presents the results of experimental design no. 3, which dealt with the effects of w/c-ratio, cement type and curing condition on the compressive strength. The results of tests on 18 batches shown in Table C.2 in Appendix C were used for analysis in a manner similar to that done for the previous two designs. Mix parameters and batch numbers for all 18 batches can be found in Table 6.4. Comparison of mean compressive strengths of concretes using three different levels of w/c-ratio and three types of cement at the six curing conditions are presented in Table 8.11.

ANOVA was performed on the test results by using the linear model as shown by Equation 6.3. Results of ANOVA are summarized in Table 8.12. It can be seen from Table 8.12 that all three main effects, namely w/c-ratio (W), cement type (C), and curing condition (D) and the effects of interactions WC and CD are significant while the interactions WC and WCD are not significant. It, therefore, follows that:

1. Since w/c-ratio interacts with cement type (and not curing condition), comparison of the effects of w/c-ratios must be made separately for each cement type, averaged over the six curing conditions.

Table 8.11 Mean Compressive Strengths of Concrete in Design No. 3

W/C-Ratio	Cement Type	Compressive Strength (psi)					
		Curing Condition*					
		1	2	3	4	5	6
0.45	I	5158	5663	7503	7256	7372	8028
	II	4680	5403	7263	6738	6543	7834
	III	6000	5984	7407	7092	7133	7430
0.38	I	5649	6287	7589	7704	7640	8324
	II	5505	6094	7602	7287	7317	8140
	III	7232	7516	8628	8339	8307	8839
0.33	I	6744	7117	8933	8760	8797	9440
	II	6533	6656	8459	8058	8152	9081
	III	8501	8376	10085	9805	9787	10182

Note: * See footnotes on Table 8.1

Table 8.12 Result of ANOVA on Compressive Strength for Design No. 3

Factor	df	MS	F-Value	P
W	2	61065913.2	80.90	0.0001*
C	2	21346782.5	28.28	0.0001*
WC	4	3038141.8	4.02	0.0385*
B(WC)	9	754846.1	-	-
D	5	31839695.8	249.10	0.0001*
WD	10	113737.4	0.89	0.5497
CD	10	705633.4	5.52	0.0001*
WCD	20	44377.6	0.35	0.9937
BD(WC)	45	127820.3	-	-
e	108	24940.96		

Note: * Significant at α level of 0.05

2. Since cement type interacts with both w/c-ratio and curing condition, comparison of the effects of cement types must be made separately for each w/c-ratio and curing condition.
3. Since curing condition interacts with cement type (and not w/c-ratio), comparison of the effects of curing conditions must be made separately for each cement type, averaged over the three w/c-ratios.

The comparison of means was done by Duncan's multiple-range test at an α level of 0.05. Table 8.13 shows the comparison of the effects of w/c-ratios for various cement types. It can be seen that concretes with w/c-ratio of 0.33 produced significantly higher compressive strength than those with w/c-ratio of 0.38 or 0.45 for all three cement types. Concretes with w/c-ratio of 0.38 gave higher compressive strength than those with w/c-ratio of 0.45 for all cement types, but the difference is statistically not significant for cement Types I and II. The effects of w/c-ratio on compressive strength as observed in this design are similar to those in design no. 2.

Table 8.14 shows the comparison of the effects of cement types for various combinations of w/c-ratios and curing conditions. It shows that cement type has some effects on the compressive strength, especially at the early ages of concrete. However, for the 90-MC curing condition, the effect is less pronounced. Concretes using cement Type III exhibited higher compressive strength than concretes using cement Type I or II for the majority of the cases. The compressive strengths of concretes using cement Types I and II are not significantly different from each other at α level of 0.05 for all cases except one (w/c of 0.45 and

Table 8.13 Grouping of W/C-Ratio by Cement Type on the Basis of Compressive Strength from Duncan's Test (Design No. 3)

Cement Type	W/C-Ratio	Means (psi)	Grouping
I	0.33	8299	A
	0.38	7198	B
	0.45	6830	B
II	0.33	7823	A
	0.38	6991	B
	0.45	6411	B
III	0.33	9456	A
	0.38	8143	B
	0.45	6840	C

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

Table 8.14 Grouping of Cement Type by W/C-Ratio and Curing Condition on the Basis of Compressive Strength from Duncan's Test (Design No. 3)

W/C-Ratio	Cement Type	Duncan's Grouping for Each Curing Condition					
		AC*	CC	SC	7-MC	28-MC	90-MC
0.33	I	B (8760)	B (8797)	B (6744)	B (7117)	B (8933)	B (9440)
	II	B (8058)	B (8152)	B (6533)	B (6656)	B (8459)	B (9081)
	III	A (9805)	A (9787)	A (8501)	A (8376)	A (10085)	A (10182)
0.38	I	AB (7704)	AB (7640)	B (5649)	B (6287)	B (7589)	A (8324)
	II	B (7287)	B (7317)	B (5505)	B (6094)	B (7602)	A (8140)
	III	A (8339)	A (8307)	A (7232)	A (7516)	A (8628)	A (8839)
0.45	I	A (7256)	A (7372)	B (5158)	A (5663)	A (7503)	A (8028)
	II	A (6738)	B (6553)	B (4680)	A (5403)	A (7263)	A (7834)
	III	A (7092)	AB (7133)	A (6000)	A (5984)	A (7406)	A (7430)

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

** Mean of compressive strength (psi)

CC curing condition). The effects of cement type as observed in this design are also in agreement with those in design no. 1.

Table 8.15 displays the comparison of the effects of curing condition for various cement types. It can be seen that the 90-day moist-cured specimens produced the highest compressive strength for all cement types. The 28-MC curing condition resulted in slightly higher compressive strength than the AC and CC curing conditions, but the difference is not significant at an α level of 0.05. The 7-MC and SC curing conditions produced the same compressive strength for concretes using cement Type III, but for concretes using cement Types I and II, the 7-MC curing condition produced higher compressive strength than the SC curing condition by about 8%. The effects caused by curing condition on compressive strength as observed in this design show trends that are similar to those from the previous two designs.

8.2.4 Results of Experimental Design No. 4 (Fly Ash Concrete)

The results of experimental design no. 4, which dealt with the effects of w/c-ratio, fly ash and curing condition on the compressive strength are presented in this section. The results of tests on 12 batches were analyzed in a manner similar to the previous three designs. Mix parameters and batch numbers for all 12 batches are presented in Table 6.5. Comparisons of mean compressive strength between the control concrete and fly ash concrete are presented in Table 8.16. Figures 8.1 and 8.2 show plots of mean compressive strength as functions of water-cementitious ratio for 28-day and 90-day moist curing, respectively. As can be seen from these two figures and Table 8.16, the compressive strengths of the control concrete are higher than those of the fly ash

Table 8.15 Grouping of Curing Condition by Cement Type on the Basis of Compressive Strength from Duncan's Test (Design No. 3)

Cement Type	Curing Condition*	Means (psi)	Grouping
I	90-MC	8597	A
	28-MC	8008	B
	AC	7906	B
	CC	7937	B
	7-MC	6355	C
	SC	5850	D
II	90-MC	8351	A
	28-MC	7775	B
	AC	7361	B
	CC	7341	B
	7-MC	6051	D
	SC	5573	E
III	90-MC	8817	A
	28-MC	8706	A
	AC	8412	A
	CC	8409	A
	7-MC	7292	B
	SC	7244	B

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

Table 8.16 Mean Compressive Strength of Control Concrete
and Fly Ash Concrete

Percent Fly Ash	W/(C+F) Ratio	Compressive Strength (psi)					
		Curing Condition*					
		1	2	3	4	5	6
0	0.45	4680	5403	7263	6738	6553	7834
0	0.38	5505	6094	7602	7287	7317	8140
0	0.33	6533	6656	8459	8058	8152	9081
20	0.45	3885	4299	6205	5859	5495	7423
20	0.38	4641	4864	6679	6263	6224	7614
20	0.33	6508	6379	8309	7938	7681	8686

Note: * See footnotes on Table 8.1

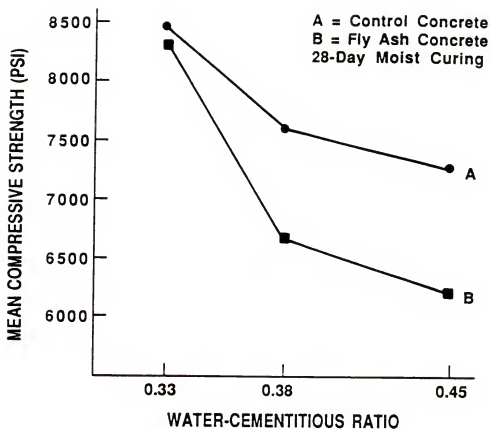


Figure 8.1 Effects of Water-Cementitious Ratio on the Compressive Strength of Control and Fly Ash Concretes at 28-Day Moist Curing

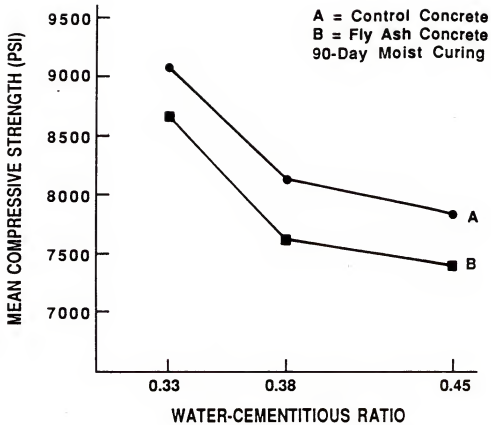


Figure 8.2 Effects of Water-Cementitious Ratio on the Compressive Strength of Control and Fly Ash Concretes at 90-Day Moist Curing

concrete. At 90-day moist curing, the compressive strength of fly ash concrete is lower than that of the control concrete by about 5%.

ANOVA was performed on the compressive strength data by using linear model as shown in Equation 6.4. Results of ANOVA are summarized in Table 8.17. From Table 8.17, it is seen that all three main effects, namely fly ash (F), w/c-ratio (W), and curing condition (D) are significant. None of the interaction effects are significant. It, therefore, follows that:

1. Since fly ash (F) does not interact with other two factors, the fly ash can be evaluated by comparing the means of compressive strength for two levels of percent of fly ash, averaged over the other two factors.
2. Since w/c-ratio (W) does not interact with the other two factors, the w/c-ratio can be evaluated by comparing the means of compressive strength for three w/c-ratios, averaged over the other two factors.
3. Since curing condition (D) does not interact with the other two factors, the relative effects caused by the six curing conditions can be assessed by comparing the means of compressive strength for six curing conditions, averaged over the other two factors.

The Duncan's multiple-range test at a level of significance of 0.05 was used to compare the means of compressive strength. The results are summarized in Tables 8.18 through 8.20. It can be seen from Table 8.18 that the mean compressive strength of fly ash concrete (averaged over three w/c-ratios and six curing conditions) is lower than that of the control concrete by about 10%.

Table 8.17 Result of ANOVA on Compressive Strength for Design No. 4

Factor	df	MS	F-Value	P
F	1	17085822.3	8.33	0.0278*
W	2	37719941.2	18.39	0.0028*
FW	2	1828250.6	0.89	0.4582
B(FW)	6	2051048.2	-	-
D	5	28130514.2	129.86	0.0001*
FD	5	172726.9	0.80	0.5602
WD	10	231704.8	1.07	0.4145
FWD	10	69542.0	0.32	0.9691
BD(FW)	30	216614.0	-	-
ϵ	72	22885.3		

Note: * Significant at α level of 0.05

Table 8.18 Grouping of Percent of Fly Ash on the Basis of Compressive Strength from Duncan's Test (Design No. 4)

Percent Fly Ash	Mean (psi)	Grouping
0	7075	A
20	6386	B

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

Table 8.19 Grouping of W/C-Ratio on the Basis of Compressive Strength from Duncan's Test (Design No. 4)

W/C-Ratio	Means (psi)	Grouping
0.33	7703	A
0.38	6519	B
0.45	5970	B

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

Table 8.20 Grouping of Curing Condition on the Basis of Compressive Strength from Duncan's Test (Design No. 4)

Curing Condition*	Means (psi)	Grouping
90-MC	8130	A
28-MC	7420	B
AC	7024	C
CC	6904	C
7-MC	5616	D
SC	5292	E

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

Table 8.19 shows that concretes with a w/c-ratio of 0.33 produced significantly higher compressive strength than those with a w/c-ratio of 0.38 or 0.45. Concretes with a w/c-ratio of 0.38 gave slightly higher compressive strength than those with a w/c-ratio of 0.45, but they were not significantly different from each other. These results are in agreement with results from design nos. 2 and 3.

From Table 8.20, it can be seen that the continued moist curing tends to increase the compressive strength. The 90-day moist curing resulted in the highest compressive strength. The 28-MC curing condition produced higher compressive strength than the AC and CC curing conditions. The compressive strengths of specimens with AC and CC curing conditions were about the same. The 7-MC curing condition produced higher compressive strength than the SC curing condition by about 6%. Again, the effects of curing condition on the compressive strength as observed in this design are also in agreement with those in the previous three designs for the majority of the cases.

8.2.5 Results of Experimental Design No. 5

This section presents the results of experimental design no. 5, which dealt with the effects of maximum aggregate size and curing conditions on the compressive strength. The results of tests on the six batches from Table C.2 in Appendix C were analyzed. Mix parameters and batch numbers for all six batches can be found in Table 6.6. The mean compressive strengths of concretes in this design using different maximum aggregate sizes at six curing conditions are presented in Table 8.21. It can be seen that the maximum aggregate size of 1 inch produced slightly lower strength than the maximum aggregate size of $\frac{3}{4}$ inch or $\frac{3}{8}$ inch.

Table 8.21 Mean Compressive Strengths of Concrete in Design No. 5

Maximum Aggregate Size (inches)	Compressive Strength (psi)					
	Curing Condition*					
	1	2	3	4	5	6
3/8	6861	7001	9204	8743	8664	9900
3/4	6744	7118	8933	8760	8797	9440
1	6678	6715	8353	8082	7962	9067

Note: * See footnotes on Table 8.1

ANOVA was performed on the compressive strength data by using the mathematical model as shown in Equation 6.5. Results of ANOVA are summarized in Table 8.22. It can be noted that the effects of curing condition (D) and the interaction of maximum aggregate size and curing condition (MD) are significant while the effects on maximum aggregate size (M) is insignificant. It, therefore, follows that:

1. Since maximum aggregate size interacts with curing condition, comparison of means of compressive strength for three maximum aggregate sizes can be made separately for each curing condition.
2. Since curing condition interacts with maximum aggregate size, comparison of means of compressive strength for six curing conditions must be made separately for each maximum aggregate size.

A comparison of means for maximum aggregate size, for each curing condition, was performed using Duncan's multiple-range tests at a level of significance of 0.05. The results are summarized in Table 8.23. From Table 8.23, it can be seen that concretes with a maximum aggregate size of 1 inch produced significantly lower compressive strength than those with a maximum aggregate size of $\frac{3}{4}$ inch or $\frac{3}{8}$ inch for the AC, CC, 28-MC and 90-MC curing conditions. All three maximum aggregate sizes exhibited about the same compressive strength for the 7-MC and SC curing conditions. The maximum aggregate sizes of $\frac{3}{4}$ inch and $\frac{3}{8}$ inch resulted in the same compressive strength for all curing conditions.

Similarly, a comparison of means for curing condition, for each maximum aggregate size, was performed by using Duncan's test, at the same level of significance. These results are presented in Table 8.24.

Table 8.22 Result of ANOVA on Compressive Strength for Design No. 5

Factor	df	MS	F-Value	P
M	2	2370218.6	4.42	0.1274
B(M)	3	535820.4	-	-
D	5	13991579.1	364.67	0.0001*
MD	10	138869.3	3.62	0.0011*
ϵ	51	38367.9		

Note: * Significant at α level of 0.05

Table 8.23 Grouping of Maximum Aggregate Size by Curing Condition on the Basis of Compressive Strength from Duncan's Test (Design No. 5)

Curing Condition*	Maximum Aggregate Size (inches)	Means (psi)	Grouping
90-MC	3/8	9899	A
	3/4	9440	A B
	1	9066	B
28-MC	3/8	9204	A
	3/4	8933	A
	1	8353	B
7-MC	3/4	7117	A
	3/8	7001	A
	1	6715	A
AC	3/4	8760	A
	3/8	8743	A
	1	8081	B
CC	3/4	8798	A
	3/8	8663	A
	1	7962	B
SC	3/8	6861	A
	3/4	6744	A
	1	6678	A

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

Table 8.24 Grouping of Curing Condition by Maximum Aggregate Size on the Basis of Compressive Strength from Duncan's Test (Design No. 5)

Maximum Aggregate Size (inches)	Curing Condition*	Means (psi)	Grouping
$\frac{3}{8}$	90-MC	9899	A
	28-MC	9204	B
	AC	8743	C
	CC	8663	C
	7-MC	7001	D
	SC	6861	D
$\frac{3}{4}$	90-MC	9440	A
	28-MC	8933	B
	AC	8760	B
	CC	8798	B
	7-MC	7117	C
	SC	6744	C
1	90-MC	9066	A
	28-MC	8353	B
	AC	8081	B
	CC	7962	B
	7-MC	6715	C
	SC	6678	C

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

It shows that the 90-day moist curing produced the highest compressive strength, and the AC and CC curing conditions produced about the same compressive strength for all maximum aggregate sizes. The 28-MC curing condition resulted in higher compressive strength than the AC and CC curing conditions. The SC curing condition gave slightly lower compressive strength than the 7-MC curing condition, but they were not significantly different from each other at a level of 0.05 for all cases. Again, the effects of curing condition on compressive strength as observed in this design show trends which are similar to those in the previous four designs.

8.2.6 Results of Experimental Design No. 6 (Silica Fume Concrete)

The results of experimental design no. 6, which dealt with the effects of silica fume and curing condition on the compressive strength are presented in this section. The results of tests on two batches of control concrete as shown in Table C.2 and eight batches of silica fume concrete as shown in Tables C.10 and C.11 in Appendix C were analyzed in a manner similar to that done for design no. 5. Mix parameters and batch numbers for all 10 batches are presented in Table 6.8. Figure 8.3 shows the plots of the mean compressive strengths as functions of percent addition of silica fume. It can be seen that the compressive strength increases as the silica fume increases up to 10% addition of silica fume, and then decreases beyond 10% addition, at both 28-day and 90-day moist curing. The addition of silica fume increases the compressive strength by about 7% to 11% and 4% to 8% of control concrete at 28-day and 90-day moist curing, respectively.

ANOVA was performed on the test results by using a linear model as shown in Equation 6.6. Results of ANOVA are presented in Table 8.25.

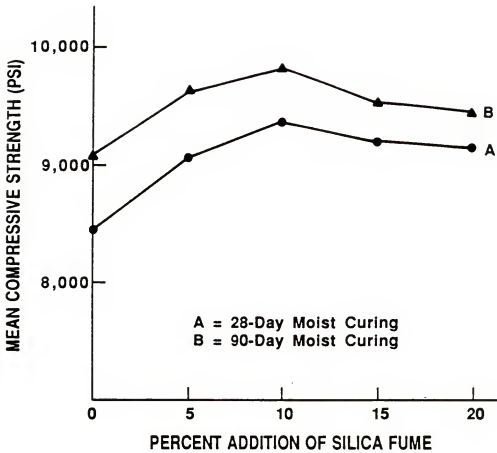


Figure 8.3 Effects of Silica Fume Addition on the Compressive Strength of Concrete with a Water-Cementitious Ratio of 0.33 and Brooksville Aggregate

Table 8.25 Result of ANOVA on Compressive Strength for Design No. 6

Factor	df	MS	F-Value	P
S	4	748470.2	3.71	0.0914
B(S)	5	201646.5	-	
D	1	2067520.9	51.0	0.0001*
SD	4	37404.6	0.92	0.4665
e	25	40544.3		

Note: * Significant at α level of 0.05

It can be seen that the effect of curing condition (D) is significant while the other factors are not significant.

Since the interaction between percent addition of silica fume and curing condition is insignificant, the comparison of means of one main factor, averaged over the other main factor, can be made.

The comparison of means for percent addition of silica fume averaged over two curing conditions was performed by using Duncan's test at a level of significance of 0.05. This was done to further clarify the results of the ANOVA. The results are summarized in Table 8.26. It can be seen that the control concrete produced lower compressive strength than the silica fume concretes. The compressive strength of concrete with 10% addition is slightly higher than those with 5%, 15%, and 20% additions, but they are not significantly different from one another at an α level of 0.05.

The effect of curing condition is significant. The compressive strength of specimens cured 90 days in the moist room is significantly higher than specimens cured 28 days in the moist room.

8.2.7 Results of Silica Fume Concrete Made with Calera Aggregate

The results of compressive strength tests on the silica fume concretes made with Calera aggregate are presented in this section. The test results of two batches of silica fume concretes made with Calera aggregate (10% and 20% additions of silica fume) are presented in Tables C.10 and C.11 while the test results of the control concrete are presented in Table C.2 in Appendix C (see batch numbers 13 and 40).

Plots of percent addition of silica fume vs. mean compressive strengths for two curing conditions are shown in Figure 8.4. It can be seen that, at 28-day moist curing, the concrete reached its maximum

Table 8.26 Grouping of Percent Addition of Silica Fume on the Basis of Compressive Strength from Duncan's Test (Design No. 6)

Percent Addition of Silica Fume	Means (psi)	Grouping
10	9598	A
15	9369	A
5	9349	A
20	9300	A
0	8770	B

Note: - Means with the same letter are not significantly different
($\alpha = 0.05$)

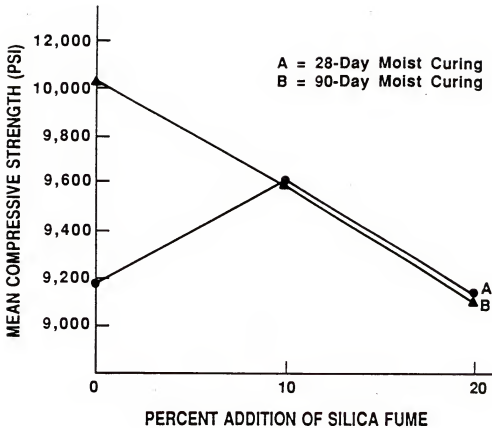


Figure 8.4 Effects of Silica Fume Addition on the Compressive Strength of Concrete with a Water-Cementitious Ratio of 0.33 and Calera Aggregate

compressive strength at 10% addition of silica fume. However, at 90-day moist curing, the maximum compressive strength occurred when no silica fume was added. Due to the limited number of tests on which this observation is based, further testing is needed to substantiate this observation.

8.3 Discussion of the Results

The results of a laboratory study on compressive strength have been presented in the previous sections. In this section, these results are discussed in accordance with the objectives of this study.

8.3.1 Effect of Aggregate Type

Aggregate type has some effects on the compressive strength of concrete. Under the same conditions, concretes using Calera aggregate had higher compressive strength than concretes using Brooksville or river gravel aggregate. This was due to the fact that the bond between the aggregate and hydrated cement paste for a rough and angular crushed rock (Calera aggregate) is higher than a smooth and round aggregate (river gravel). Jones and Kaplan [38] found that the mechanical interlocking of the coarse aggregate contributes to the ultimate strength of concrete in compression. The mechanical interlocking of Calera aggregate is also higher than the mechanical interlocking of river gravel. The high strength of Calera aggregate is a factor for the higher compressive strength as compared with the Brooksville aggregate.

8.3.2 Effect of Cement Type

The type of cement used also affects the compressive strength. For concrete age of up to 90 days in the moist room, Type III cement produced higher compressive strength than Type I and II cements. Type III

cement is a high early strength cement. The fineness of cement Type III is higher than the other two cement types. The fineness of cement is the main reason why concretes using cement Type III produced higher compressive strength than those using cement Type I or II, especially at concrete ages of up to about 1 month. At 90-day moist curing, the cement type has less effect on compressive strength. This is because the degree of hydration for all three cement types reaches about the same level ultimately.

Type II cement is moderate sulfate resistant and has a slower rate of hydration than Type I cement. The test results from this study indicates that Type I cement produced a higher compressive strength than Type II cement for the majority of the cases, especially for the first five curing conditions, namely AC, CC, SC, 7-MC and 28-MC. In some cases, the compressive strengths of these two cement types are not significantly different from each other at an α level of 0.05. This observation can be clearly seen in Tables 8.4 and 8.14.

8.3.3 Effect of W/C-Ratio

The w/c-ratio is one of the major parameters affecting the compressive strength. In all three designs, concretes with lower w/c-ratios exhibited higher compressive strength than those with higher w/c-ratios. Concretes with a w/c-ratio of 0.33 produced significantly higher compressive strength than those with w/c-ratios of 0.38 and 0.45. The compressive strengths of concretes with a w/c-ratio of 0.38 are generally higher than those with a w/c-ratio of 0.45, but in some cases, they are not significantly different from each other.

8.3.4 Effect of Maximum Aggregate Size

The effect of maximum aggregate size on compressive strength is less pronounced than the effects of the three parameters previously discussed. Mindess and Young [40] stated that the use of larger maximum aggregate size reduces the bond strength between the aggregate and cement paste and also provides more restraint on volume changes in the paste, which can cause stresses in the paste. These two factors are believed to be the main reasons why the larger maximum aggregate size tends to produce lower compressive strength. In this study, the 1 inch maximum aggregate size produced lower compressive strength than the $3/8$ -inch or $3/4$ -inch maximum aggregate size by about 9% or less. The maximum aggregate sizes of $3/8$ inch and $3/4$ inch gave about the same compressive strength. It can be observed that the three maximum aggregate sizes used in this study are within a narrow range. Therefore, the effect of maximum aggregate size on the compressive strength of concrete was not too pronounced.

8.3.5 Effect of Fly Ash

The use of fly ash as a cement replacement affects the compressive strength of concrete. As can be seen from Table 8.16 and Figures 8.1 and 8.2, the compressive strength of the control concrete is higher than those of the fly ash concrete. This is because the Class F fly ash used in this study develops strength at a relatively slow rate. At early ages, the cement paste of fly ash concrete has more pores than the control concrete. The strength of the mortar is one of the major factors that contributes to the compressive strength of concrete. Therefore, the fly ash concrete, which has a lower mortar strength exhibited lower compressive strength than the control concrete. At later ages, the fly

ash reacts with calcium hydroxide to produce calcium silicate hydrate which can reduce the pore space and make a denser cement paste. The compressive strengths of fly ash and control concrete are expected to be approximately the same [31]. In this study, the compressive strength of fly ash concrete at 90-day moist curing is lower than that of the control concrete by about 5% for all w/c-ratios.

8.3.6 Effect of Silica Fume

8.3.6.1 Silica fume concrete made with Brooksville aggregate. The addition of silica fume to concretes made with Brooksville aggregate increased the compressive strength by about 7% to 11% and 4% to 8% of control concrete at 28-day and 90-day moist curing, respectively. This is because the added silica fume reduces the volume of large pores and capillaries in the cement pastes [28]. The 10% addition of silica fume produced the highest compressive strength.

Jones and Kaplan [38] found that the compressive strengths of concretes are higher than the strengths of the comparable mortars. This is because the mechanical interlocking of the coarse aggregate contributes to the higher strengths. Since the amounts of coarse aggregate in the silica fume concretes are about the same for all eight batches, the strengths of mortars would be the main factor in the difference in the compressive strengths of silica fume concrete. In this study, the addition of silica fume reduced by volume the amount of sand in the control concrete. The higher the percent addition of silica fume, the higher the replacement of sand by silica fume would be. Maximum compressive strength was observed to reach a maximum at a silica fume addition of 10%. At a silica fume addition of over 10%, the mortar may be lacking in sand and may explain the drop in compressive strength of concrete.

8.3.6.2 Silica fume concrete made with Calera aggregate. The compressive strength of concrete with 10% addition of silica fume was higher than the concrete with no addition or 20% addition of silica fume at 28-day moist curing. This agrees with the result of silica fume concrete made with Brooksville aggregate. However, at 90-day moist curing, the compressive strength of the silica fume concrete was lower than that of the control concrete. Since this observation was based on limited amount of test data, further testing is needed to substantiate this observation.

8.3.7 Effect of Curing Condition

The curing condition is also one of the major factors affecting the compressive strength. This can be clearly seen from all the ANOVA results in this chapter. The continued moist curing can reduce the pores in the cement paste and also produce a higher bond strength between the mortar and the coarse aggregate. Therefore, the 90-day moist-curing condition exhibited higher compressive strength than the other five curing conditions.

The 28-MC curing condition produced higher compressive strength than the AC, CC and SC curing conditions. It is believed that, the moist-cured specimens should have a higher degree of hydration of cement than the air-cured and steam-cured specimens and specimens with curing compound. The compressive strengths of the concretes at curing conditions of AC and CC were not different from each other at an α level of 0.05, and were at least 90% of the compressive strength of the concretes at the 28-MC curing condition. From the results in Chapter 7, the moduli of rupture of the concretes at the AC and CC curing conditions were about 70% to 85% of those at the 28-MC curing condition. It can be

inferred that the shrinkage at the surface of tested specimens has more effects on the modulus of rupture than the compressive strength.

The compressive strength of steam-cured specimen was about 65% to 85% and about 90% to 105% of the compressive strength of 28-day and 7-day moist-curing conditions, respectively. Higginson [20] found that the compressive strength of steam-cured specimens was about 80% and 50% of the compressive strength of 7-day and 28-day moist-curing conditions, respectively. An initial delay of 3 hours prior to steaming was used in the Higginson study, but in this study a 24-hour delay was used (as described in Chapter 4). This could be the main reason why the compressive strengths of steam-cured specimens from this study are higher than the results obtained in Higginson's study.

From Table 8.16, it can be seen that the compressive strengths of fly ash concrete when steam cured are 63%, 69% and 78% of those of the 28-day moist-cured specimens for water-cementitious ratios of 0.45, 0.38 and 0.33, respectively. Therefore, it indicates that the effectiveness of steam curing on fly ash concrete increases as the water-cementitious ratio decreases.

8.4 Summary of Results

The findings presented in this chapter on the effects of mix parameter and curing condition on the compressive strength are summarized as follows:

1. The w/c-ratio is the major parameter affecting the compressive strength. Generally, the higher the w/c-ratio, the lower the compressive strength would be.

2. Curing condition is also one of the major parameters affecting the compressive strength. The continued curing tends to increase the compressive strength. The 90-MC curing condition produces the highest compressive strength. The 28-MC curing condition shows higher compressive strength than the AC and CC curing conditions by up to about 10%. The compressive strengths of concretes at the AC and CC curing conditions are about the same. The SC curing condition produces about 60% to 85% of the compressive strength of the 28-MC curing condition. The 7-MC curing condition gives slightly higher or about the same compressive strength as compared with the SC curing condition.
3. The aggregate type has some effects on the compressive strength. Concretes using Calera aggregate exhibit higher compressive strength than concretes using Brooksville or river gravel aggregate. The compressive strengths of concretes using Brooksville and river gravel aggregates are about the same for all cases.
4. Cement type has some effects on the compressive strength, especially at the early age of concrete samples. Cement Type III shows higher compressive strength than cement Type I or II. The compressive strength of concretes using cement Type I is slightly higher than concretes using cement Type II for the AC, CC, SC, 7-MC and 28-MC curing conditions. At 90-day moist curing, both cement Types I and II produce about the same compressive strength.

5. The use of larger maximum aggregate sizes result in concretes with lower compressive strength than those with smaller maximum aggregate sizes. However, the compressive strengths of the three different maximum aggregate sizes are not significantly different from one another at an α level of 0.05.
6. It can be noted that the effects of w/c-ratio, curing condition, aggregate type, cement type and maximum aggregate size on the compressive strength are similar to their effects on the modulus of rupture.
7. The fly ash concrete used in this study, which is made by replacing 20% of cement by a Class F fly ash develops compressive strength very slowly. The compressive strength of the fly ash concrete is significantly lower than the control concrete at the early ages. At 90-day moist curing, the compressive strength of the fly ash concrete is lower than that of the control concrete by about 5%. The effect of the replacement of cement by fly ash on the compressive strength is significant at an α level of 0.05, while it is insignificant on the modulus of rupture.
8. The silica fume concrete made with Brooksville aggregate exhibits the highest compressive strength of a silica fume addition of approximately 10% by weight of cement.

CHAPTER 9 RESULTS OF SPLITTING TENSILE STRENGTH TESTS

9.1 Introduction

The splitting cylinder test is one of the three commonly-used methods to determine the tensile strength of concrete. This method of applying tension in the form of splitting was originally suggested by Fernando Carnerio, a Brazilian, and thus the test is often referred to as the Brazilian test [5]. The tested specimens and testing machine for this test are the same as those used for the compression test. The splitting tensile strength and the modulus of rupture are known to be linearly related [4]. The splitting tensile strength was included in this research program because the test is relatively easier to perform. One of the objectives of this testing program was to determine if the splitting tensile test could be used in place of the modulus of rupture test.

This chapter presents the results of splitting tensile strength tests for all six experimental designs. The details of all experimental designs are presented in Chapter 6. The general laboratory procedure described in Chapter 4 was used to prepare and test the cylindrical specimens. The method of analysis used is described in Chapter 6.

9.2 Analysis of Results

9.2.1 Results of Experimental Design No. 1

This section presents the results of experimental design no. 1, which dealt with the effects of aggregate type, cement type and curing condition on the splitting tensile strength. The results of tests on 18 batches, as shown in Table C.3 in Appendix C, were analyzed. Mix parameters and batch numbers for all 18 batches are displayed in Table 6.2. The mean splitting tensile strengths of the concretes in design no. 1 using three different aggregate and cement types at the six curing conditions are presented in Table 9.1.

ANOVA was performed on the test results by using the linear model as shown by Equation 6.1. Results of ANOVA are summarized in Table 9.2. The ANOVA results show that only the interaction of aggregate type and cement type (AC) are not significant while all the main effects and the other interactions are significant. It, therefore, follows that:

1. Comparison of aggregate type (A) must be made at each cement type, for each curing condition.
2. Comparison of cement type (C) must be made by aggregate type, for each curing condition.
3. Comparison of curing condition (D) must be made by each aggregate type, for each cement type.

A comparison of means for aggregate type, for each cement type and curing condition was performed using Duncan's multiple-range test at an α level of 0.05. The results are summarized in Table 9.3. It can be seen that the concretes using Calera aggregate generally exhibited significantly higher splitting tensile strength than those using Brooksville or river gravel aggregate. Concretes using Brooksville aggregate

Table 9.1 Mean Splitting Tensile Strength of Concrete in Design No. 1

		Splitting Tensile Strength (psi)					
Aggregate Type	Cement Type	Curing Condition*					
		1	2	3	4	5	6
Brooksville	I	627	622	707	545	570	734
	II	550	489	629	569	624	695
	III	641	643	707	652	651	699
Calera	I	671	707	825	756	702	859
	II	668	636	748	623	631	839
	III	690	734	764	707	715	822
River Gravel	I	526	531	635	636	603	711
	II	587	556	672	643	616	691
	III	642	650	681	633	652	752

Note: * Curing conditions:

1 = 16 hours steam curing

2 = 7 days moist curing

3 = 28 days moist curing

4 = 3 days moist curing and 25 days in air

5 = 1 day in form and 27 days with curing compound

6 = 90 days moist curing

Table 9.2 Result of ANOVA on Splitting Tensile Strength for Design No. 1

Factor	df	MS	F-Value	P
A	2	224589.3	28.24	0.0001*
C	2	57907.8	7.28	0.0131*
AC	4	15769.8	1.98	0.1808
B(AC)	9	7951.6	-	-
D	5	113382.7	94.16	0.0001*
AD	10	3654.6	3.04	0.0051*
CD	10	4733.2	3.93	0.0007*
ACD	20	5134.0	4.26	0.0001*
BD(AC)	45	1204.16	-	-
ϵ	108	1187.86		

Note: * Significant at α level of 0.05

Table 9.3 Grouping of Aggregate Type by Cement Type and Curing Condition on the Basis of Splitting Tensile Strength from Duncan's Test (Design No. 1)

Cement Type	Aggregate Type	Duncan's Grouping for Each Curing Condition					
		AC*	CC	SC	7-MC	28-MC	90-MC
I	Brooksville	C (545)	B (570)	A (627)	B (622)	B (707)	B (734)
	Calera	A (756)	A (702)	A (671)	A (707)	A (825)	A (858)
	River Gravel	B (636)	B (603)	B (526)	C (530)	C (635)	B (711)
II	Brooksville	B (569)	A (623)	C (500)	B (489)	B (629)	B (695)
	Calera	A (623)	A (631)	A (668)	A (636)	A (747)	A (839)
	River Gravel	A (643)	A (615)	B (587)	B (556)	B (672)	B (691)
III	Brooksville	AB (651)	A (651)	A (640)	B (642)	AB (707)	B (699)
	Calera	A (707)	A (714)	A (690)	A (734)	A (764)	A (823)
	River Gravel	B (633)	A (653)	A (642)	B (650)	B (682)	B (752)

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

** Mean of splitting tensile strength (psi)

generally gave the same splitting tensile strength as those using river gravel. In three combinations of cement type and curing conditions, namely, 1) cement Type II and CC; 2) cement Type III and CC; and 3) cement Type III and SC, the three aggregates are not significantly different from one another.

A comparison of means for cement type for each aggregate type and curing condition was performed using Duncan's test at an α level of 0.05. The results are presented in Table 9.4. It can be seen that cement Types I and III generally produced about the same splitting tensile strength. Concretes using cement Type II generally exhibited slightly lower splitting tensile strength than those using cement Type I or III. For concretes using river gravel, cement Types I and II gave the same splitting tensile strength. At 90-MC curing condition, the cement type had no significant effect on splitting tensile strength for all aggregate types.

Similarly, a comparison of means for curing condition, for each aggregate and cement types was performed using Duncan's test, at the same level of significance. These results are summarized in Table 9.5. It can be seen that the 90-day moist-cured specimens exhibited higher splitting tensile strength than the specimens of the other five curing conditions for all cases. The 28-MC curing condition resulted in slightly higher splitting tensile strength than the AC and CC curing conditions. The splitting tensile strength of air-cured and curing-compound specimens were not significantly different from each other for all aggregate and cement types. Also, the splitting tensile strength of steam-cured specimens was not different from that obtained from 7-day moist curing.

Table 9.4 Grouping of Cement Type by Aggregate Type and Curing Condition on the Basis of Splitting Tensile Strength from Duncan's Test (Design No. 1).

Aggregate Type	Cement Type	Duncan's Grouping for Each Curing Condition					
		AC*	CC	SC	7-MC	28-MC	90-MC
Brooksville	I	B (545)	B (570)	A (627)	A (622)	A (707)	A (734)
	II	B (569)	AB (623)	B (500)	B (489)	B (629)	A (695)
	III	A (651)	A (651)	A (640)	A (642)	A (707)	A (699)
Calera	I	A (756)	A (702)	A (671)	A (707)	A (825)	A (858)
	II	B (623)	B (631)	A (668)	B (636)	B (747)	A (839)
	III	A (707)	A (714)	A (690)	A (734)	AB (764)	A (823)
River Gravel	I	A (636)	A (603)	B (526)	B (530)	A (635)	A (711)
	II	A (643)	A (615)	AB (587)	B (556)	A (672)	A (691)
	III	A (633)	A (653)	A (642)	A (650)	A (681)	A (752)

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

** Mean of splitting tensile strength (psi)

Table 9.5 Grouping of Curing Condition by Aggregate Type and Cement Type on the Basis of Splitting Tensile Strength from Duncan's Test (Design No. 1)

Aggregate Type	Curing Condition*	Duncan's Grouping for Each Cement Type					
		I		II		III	
Brooksville	90-MC	A	** (734)	A	(695)	A	(699)
	28-MC	A	(707)	AB	(629)	A	(707)
	AC	C	(545)	BC	(569)	A	(651)
	CC	BC	(570)	AB	(623)	A	(651)
	7-MC	B	(622)	D	(489)	A	(642)
	SC	B	(627)	CD	(500)	A	(640)
Calera	90-MC	A	(858)	A	(839)	A	(823)
	28-MC	AB	(825)	B	(747)	AB	(764)
	AC	BC	(756)	C	(623)	B	(707)
	CC	CD	(702)	C	(631)	B	(714)
	7-MC	CD	(707)	C	(636)	B	(734)
	SC	D	(671)	C	(668)	B	(690)
River Gravel	90-MC	A	(711)	A	(691)	A	(752)
	28-MC	B	(635)	A	(672)	AB	(682)
	AC	B	(636)	A	(643)	B	(633)
	CC	B	(603)	AB	(615)	B	(653)
	7-MC	C	(530)	B	(556)	B	(650)
	SC	C	(526)	B	(587)	B	(642)

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

** Mean of splitting tensile strength (psi)

9.2.2 Results of Experimental Design No. 2

The results of experimental design no. 2, which dealt with the effects of aggregate type, w/c-ratio and curing condition on the splitting tensile strength are presented in this section. Results of tests on 18 batches, as shown in Table C.3 in Appendix C, were analyzed in a manner similar to that done for design no. 1. Mix parameters and batch numbers for all 18 batches can be found in Table 6.3. Comparison of mean splitting tensile strengths of concretes using three different aggregate types, three different levels of w/c-ratios and six curing conditions are presented in Table 9.6.

ANOVA was performed on the test results by using the linear model as shown in Equation 6.3. Results of ANOVA are summarized in Table 9.7. It can be seen that the effects of aggregate type (A), w/c-ratio (W), curing condition (D) and the interaction AD are significant while the interactions AW, WD and AWD are not significant. It, therefore, follows that:

1. Since aggregate type interacts with curing condition (and not w/c-ratio), comparison of the effects of aggregate types must be made separately for each curing condition, averaged over the three w/c-ratios.
2. Since w/c-ratio does not interact with either aggregate type or curing condition, comparison of the effects of w/c-ratios can be made by averaging over all aggregate types and all curing conditions.
3. Since curing condition interacts with aggregate type (and not w/c-ratio), comparison of the effects of curing conditions must

Table 9.6 Mean Splitting Tensile Strength of Concrete in Design No. 2

		Splitting Tensile Strength (psi)					
Aggregate Type	W/C-Ratio	Curing Condition*					
		1	2	3	4	5	6
Brooksville	0.45	485	492	569	538	543	591
	0.38	496	568	512	521	535	667
	0.33	627	622	708	545	570	734
Calera	0.45	528	561	607	569	568	724
	0.38	603	659	718	662	660	776
	0.33	671	707	825	756	702	859
River Gravel	0.45	469	472	560	523	495	616
	0.38	509	504	658	654	629	673
	0.33	526	531	635	636	603	711

Note: * See footnotes on Table 9.1

Table 9.7 Result of ANOVA on Splitting Tensile Strength for Design No. 2

Factor	df	MS	F-Value	P
A	2	238064.3	28.90	0.0001*
W	2	235944.5	28.64	0.0001*
AW	4	21352.2	2.59	0.1084
B(AW)	9	8239.1	-	-
D	5	119566.2	45.99	0.0001*
AD	10	7438.0	2.86	0.0076*
WD	10	2522.8	0.97	0.4817
AWD	20	4407.0	1.70	0.0712
BD(AW)	45	2599.7	-	-
ϵ	108	1140.3		

Note: * Significant at α level of 0.05

be made separately for each aggregate type, averaged over all w/c-ratios.

The Duncan's multiple-range test at an α level of 0.05 was used to compare the means of splitting tensile strength. Table 9.8 shows the comparison of the effects of aggregate type for various curing conditions. It can be seen that the concretes using Calera aggregate produced higher splitting tensile strength than those using Brooksville or river gravel aggregate for all curing conditions. The concretes using Brooksville aggregate exhibited the same splitting tensile strength as those using river gravel for all curing conditions except for the AC curing condition. The air-cured specimens of concrete using Brooksville aggregate showed lower splitting tensile strength than that using river gravel by about 12%.

Table 9.9 shows the comparison of the effects of w/c-ratio. It can be seen that the splitting tensile strength generally increases as the w/c-ratio decreases.

Table 9.10 shows the comparison of the effects of curing condition for various aggregate types. It can be seen that the 90-MC curing condition resulted in higher splitting tensile strength than the other five curing conditions. The AC and CC curing conditions produced the same splitting tensile strength, but they produced slightly lower strength than the 28-MC curing condition. The 7-MC curing condition produced the same splitting tensile strength as the SC curing condition.

9.2.3 Results of Experimental Design No. 3

The results of experimental design no. 3, which dealt with the effects of w/c-ratio, cement type and curing condition on the splitting tensile strength are presented in this section. The results of tests on

Table 9.8 Grouping of Aggregate Type by Curing Condition on the Basis of Splitting Tensile Strength from Duncan's Test (Design No. 2)

Curing Condition*	Aggregate Type	Means (psi)	Grouping
AC	Calera	662	A
	Brooksville	534	B
	River Gravel	604	A
CC	Calera	643	A
	Brooksville	549	B
	River Gravel	576	B
SC	Calera	600	A
	Brooksville	535	B
	River Gravel	501	B
7-MC	Calera	642	A
	Brooksville	560	B
	River Gravel	502	B
28-MC	Calera	716	A
	Brooksville	596	B
	River Gravel	618	B
90-MC	Calera	786	A
	Brooksville	664	B
	River Gravel	666	B

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

Table 9.9 Grouping of W/C-Ratio on the Basis of Splitting
Tensile Strength from Duncan's Test (Design No. 2)

W/C-Ratio	Means (psi)	Grouping
0.33	665	A
0.38	611	B
0.45	550	C

Note: - Means with the same letter are not significantly different
($\alpha = 0.05$)

Table 9.10 Grouping of Curing Condition by Aggregate Type on the Basis of Splitting Tensile Strength from Duncan's Test (Design No. 2)

Aggregate Type	Curing Condition*	Means (psi)	Grouping
Brooksville	90-MC	664	A
	28-MC	596	B
	AC	534	B
	CC	549	B
	7-MC	560	B
	SC	535	B
Calera	90-MC	786	A
	28-MC	716	B
	AC	662	B C
	CC	643	C
	7-MC	642	C
	SC	600	C
River Gravel	90-MC	666	A
	28-MC	618	A B
	AC	604	A B
	CC	576	B
	7-MC	502	C
	SC	501	C

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

18 batches, as shown in Table C.3 in Appendix C, were analyzed in a manner similar to that done for the previous two designs. Mix parameters and batch numbers for all 18 batches are presented in Table 6.4. Comparison of mean splitting tensile strength of concretes using three different levels of w/c-ratios and three types of cement for the six curing conditions are presented in Table 9.11.

ANOVA was performed on the splitting tensile strength data by using the linear model as shown by Equation 6.3. Results of ANOVA are summarized in Table 9.12. It can be seen that the effects of w/c-ratio (W), cement type (C), curing condition (D) and the interaction CD are significant while the interactions WC, WD and WCD are insignificant. Therefore, the comparison of means should be made as follows:

1. Since w/c-ratio does not interact with the other two factors, the effects of w/c-ratios can be evaluated by comparing the means of splitting tensile strength for three levels of w/c-ratios, averaged over the other two factors.
2. Since cement type interacts with curing condition (and not w/c-ratio), the effects of cement types can be evaluated by comparing the means for three cement types, averaged over the aggregate types at each curing condition.
3. Since curing condition interacts with cement type (and not w/c-ratio), the effects caused by the six curing conditions can be assessed by comparing the means of splitting tensile strength for six curing conditions, averaged over the w/c-ratios at each curing condition.

Table 9.11 Mean Splitting Tensile Strength of Concrete in Design No. 3

		Splitting Tensile Strength (psi)					
W/C-Ratio	Cement Type	Curing Condition*					
		1	2	3	4	5	6
0.45	I	485	492	569	539	543	591
	II	326	387	511	499	529	574
	III	490	530	572	519	522	634
0.38	I	496	568	512	521	535	667
	II	430	469	589	496	545	632
	III	598	585	606	564	587	666
0.33	I	627	622	708	545	570	734
	II	500	489	629	569	623	695
	III	641	643	707	652	651	699

Note: * See footnotes on Table 9.1

Table 9.12 Result of ANOVA on Splitting Tensile Strength for Design No. 3

Factor	df	MS	F-Value	P
W	2	225160.1	17.98	0.0007*
C	2	105433.4	8.42	0.0087*
WC	4	3803.3	0.30	0.8684
B(WC)	9	12524.3	-	-
D	5	99073.1	35.48	0.0001*
WD	10	4774.4	1.71	0.1080
CD	10	13153.2	4.71	0.0001*
WCD	20	3315.2	1.19	0.3081
BD(WC)	45	2792.6	-	-
ϵ	108	984.57		

Note: * Significant at α level of 0.05

The Duncan's multiple-range test at an α level of 0.05 was used to compare the means of splitting tensile strength. The results are summarized in Tables 9.13 through 9.15.

Table 9.13 shows the comparison of the effects of w/c-ratio. It can be seen that the concrete with a w/c-ratio of 0.33 has a significantly higher splitting tensile strength than that with w/c-ratios of 0.38 and 0.45. The concrete with a w/c-ratio of 0.38 gave slightly higher splitting tensile strengths than that with a w/c-ratio of 0.45, but statistically they are not significantly different from each other at an α level of 0.05 (due to the great scattering of the splitting tensile test results).

Table 9.14 shows the comparison of the effects of cement type for various curing conditions. It can be seen that all three types of cement exhibited about the same splitting tensile strength for all curing conditions, except the SC and 7-MC curing conditions. The splitting tensile strengths of concretes made with cement Types I and III are significantly higher than those made with cement Type II by about 20% to 25% for the SC and 7-MC curing conditions.

From Table 9.15 which shows the comparison of the effects of curing conditions for various cement types, it can be noted that the 90-day moist curing produced the highest splitting tensile strength. The 28-MC curing condition gave slightly higher splitting tensile strength than the AC and CC curing conditions, but the splitting tensile strengths of these three curing conditions are not significantly different from one another. The splitting tensile strength of steam-cured and 7-day moist-cured specimens are about the same.

Table 9.13 Grouping of W/C-Ratio on the Basis of Splitting Tensile Strength from Duncan's Test (Design No. 3)

W/C-Ratio	Means (psi)	Grouping
0.33	628	A
0.38	559	B
0.45	517	B

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

Table 9.14 Grouping of Cement Type by Curing Condition on the Basis of Splitting Tensile Strength from Duncan's Test (Design No. 3)

Curing Condition*	Cement Type	Means (psi)	Grouping
AC	I	534	A
	II	521	A
	III	578	A
CC	I	549	A
	II	566	A
	III	586	A
SC	I	535	A
	II	419	B
	III	576	A
7-MC	I	560	A
	II	448	B
	III	586	A
28-MC	I	596	A
	II	576	A
	III	628	A
90-MC	I	664	A
	II	633	A
	III	666	A

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

Table 9.15 Grouping of Curing Condition by Cement Type on the Basis of Splitting Tensile Strength from Duncan's Test (Design No. 3)

Cement Type	Curing Condition*	Means (psi)	Grouping
I	90-MC	664	A
	28-MC	596	B
	AC	534	B
	CC	549	B
	7-MC	560	B
	SC	535	B
II	90-MC	633	A
	28-MC	576	A B
	AC	521	B
	CC	567	B
	7-MC	448	C
	SC	419	C
III	90-MC	666	A
	28-MC	628	A B
	AC	578	B
	CC	586	B
	7-MC	586	B
	SC	576	B

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

9.2.4 Results of Experimental Design No. 4 (Fly Ash Concrete)

This section presents the results of experimental design no. 4, which dealt with the effects of w/c-ratio, fly ash and curing condition on the splitting tensile strength. Results of tests on 12 batches, as shown in Table C.3 in Appendix C, were analyzed in a manner similar to the previous three designs. Mix parameters and batch numbers for all 12 batches are shown in Table 6.5. The mean splitting tensile strength of the control concrete and fly ash concrete are presented in Table 9.16.

Figures 9.1 and 9.2 show plots of the mean splitting tensile strengths as functions of water-cementitious ratio for 28-day and 90-day moist curing, respectively. It can be seen from these two figures that the splitting tensile strengths of the fly ash concrete are very close to those of the control concrete.

ANOVA was performed on the test results by using the linear model shown in Equation 6.4. Results of ANOVA on the splitting tensile strength results are summarized in Table 9.17. It can be seen that the effects of w/c-ratio (W), curing condition (D) and the interaction between fly ash replacement and curing condition (FD) are significant. The insignificant terms are fly ash (F) and the interactions FW, WD and FWD. The comparison of means must be done as follows:

1. Since fly ash interacts with curing condition and does not interact with w/c-ratio, the effects of fly ash can be evaluated by comparing the means of splitting tensile strength, averaged over the w/c-ratios at each curing condition.
2. Since w/c ratio does not interact with the other two factors, the effects of w/c-ratios can be evaluated by comparing the

Table 9.16 Mean Splitting Tensile Strength of Control Concrete and Fly Ash Concrete

Percent Fly Ash	W/(C+F) Ratio	Splitting Tensile Strength (psi)					
		Curing Condition*					
		1	2	3	4	5	6
0	0.45	326	387	511	499	529	574
0	0.38	430	469	589	496	545	632
0	0.33	500	489	629	569	623	695
20	0.45	384	445	530	463	449	601
20	0.38	453	483	582	516	500	639
20	0.33	533	512	607	567	568	686

Note: * See footnotes on Table 9.1

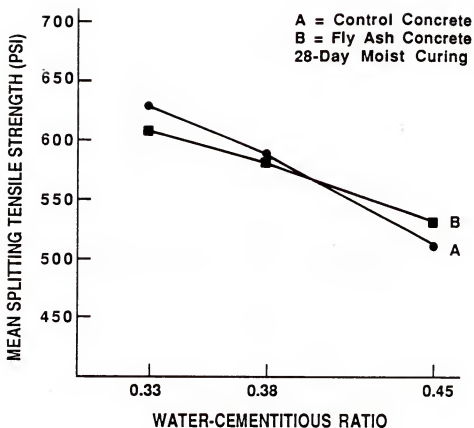


Figure 9.1 Effects of Water-Cementitious Ratio on the Splitting Tensile Strength of Control and Fly Ash Concretes at 28-Day Moist Curing

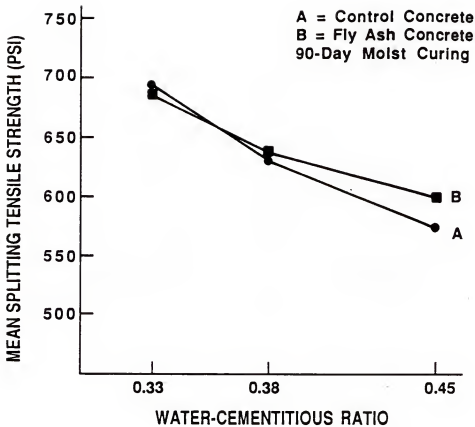


Figure 9.2 Effects of Water-Cementitious Ratio on the Splitting Tensile Strength of Control and Fly Ash Concretes at 90-Day Moist Curing

Table 9.17 Result of ANOVA on Splitting Tensile Strength for Design No. 4

Factor	df	MS	F-Value	P
F	1	81.0	0.01	0.9358
W	2	136640.6	11.88	0.0082*
FW	2	490.8	0.04	0.9585
B(FW)	6	11499.9	-	-
D	5	127954.3	47.83	0.0001*
FD	5	7405.5	2.77	0.0359*
WD	10	2290.9	0.86	0.5813*
FWD	10	986.0	0.37	0.9508
BD(FW)	30	2675.1	-	-
ϵ	72	883.7		

Note: * Significant at α level of 0.05

means of splitting tensile strength for three levels of w/c-ratios, averaged over the other two factors.

3. Since curing condition interacts with fly ash and does not interact with w/c-ratio, the effects caused by the six curing conditions can be assessed by comparing the means of splitting tensile strength for six curing conditions, averaged over the w/c-ratio at each percent replacement of fly ash.

The comparison of means of splitting tensile strengths was done by using Duncan's multiple-range test at an α level of 0.05. The results are summarized in Tables 9.18 through 9.20.

Table 9.18 shows the comparison of the effects of fly ash replacement for various curing conditions. It can be seen that fly ash concrete is not significantly different from the control concrete for all six curing conditions. However, it can be observed that the control concrete has substantially higher tensile strength than the fly ash concrete at 7-MC, CC and SC curing conditions.

Table 9.19 shows the comparison of the effects of w/c-ratio. It can be seen that the concrete with a w/c-ratio of 0.33 has a higher splitting tensile strength than that with w/c-ratios of 0.38 and 0.45, but statistically it is not significantly different from that with a w/c-ratio of 0.38. The concretes with w/c-ratios of 0.38 and 0.45 produced splitting tensile strengths which are statistically not significantly different from one another.

Table 9.20 shows the comparison of the effects of curing conditions for two levels of percent replacement of fly ash. The effect of curing is more significant for the fly ash concrete than for the control concrete. The 90-MC condition produced significantly higher splitting

Table 9.18 Grouping of Percent Replacement of Fly Ash by Curing Condition on the Basis of Splitting Tensile Strength from Duncan's Test (Design No. 4)

Curing Condition*	Percent Fly Ash	Means (psi)	Grouping
AC	0	521	A
	20	515	A
CC	0	565	A
	20	506	A
SC	0	456	A
	20	419	A
7-MC	0	448	A
	20	480	A
28-MC	0	576	A
	20	573	A
90-MC	0	633	A
	20	642	A

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

Table 9.19 Grouping of W/C-Ratio on the Basis of Splitting
Tensile Strength from Duncan's Test (Design No. 4)

W/C-Ratio	Means (psi)	Grouping
0.33	581	A
0.38	528	A B
0.45	475	B

Note: - Means with the same letter are not significantly different
($\alpha = 0.05$)

Table 9.20 Grouping of Curing Condition by Percent Replacement of Fly Ash on the Basis of Splitting Tensile Strength from Duncan's Test (Design No. 4)

Percent Fly Ash	Curing Condition*	Means (psi)	Grouping
0	90-MC	633	A
	28-MC	576	A B
	AC	521	B
	CC	566	B
	7-MC	448	C
	SC	419	C
20	90-MC	642	A
	28-MC	573	B
	AC	515	B C
	CC	506	C
	7-MC	480	C
	SC	456	C

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

tensile strength than the 28-MC condition for the fly ash concrete, while their difference was insignificant for the control concrete. The 28-MC condition produced slightly higher splitting tensile strength than the AC and CC curing conditions. The splitting tensile strengths of specimens with the AC and CC curing conditions were not significantly different from each other at an α level of 0.05. The SC and 7-MC curing conditions resulted in the same splitting tensile strength.

9.2.5 Results of Experimental Design No. 5

The results of experimental design no. 5, which dealt with the effects of maximum aggregate size and curing condition on the splitting tensile strength are presented in this section. The results of tests on six batches, as shown in Table C.3 in Appendix C, were used in the analysis for this design. Mix parameters and batch numbers for all six batches are presented in Table 6.6. Table 9.21 presents the mean splitting tensile strengths of concretes using three different maximum aggregate sizes at the six curing conditions.

ANOVA was performed on the splitting tensile strength data by using the linear model shown in Equation 6.5. Results of ANOVA are summarized in Table 9.22. It can be seen that only the effect of curing condition (D) is significant. It, therefore, follows that:

1. Since maximum aggregate size does not interact with the curing condition, the effects of maximum aggregate size can be evaluated by comparing the means of splitting tensile strength for three maximum aggregate sizes, averaged over the curing condition. Although the effect of maximum aggregate size is insignificant, the comparison of means is done to further clarify the results of the ANOVA.

Table 9.21 Mean Splitting Tensile Strength of Concrete in Design No. 5

Maximum Aggregate Size (inches)	Splitting Tensile Strength (psi)					
	Curing Condition*					
	1	2	3	4	5	6
3/8	618	647	693	611	620	721
3/4	627	622	708	545	570	734
1	568	572	640	578	579	676

Note: * See footnotes on Table 9.1

Table 9.22 Result of ANOVA on Splitting Tensile Strength for Design No. 5

Factor	df	MS	F-Value	P
M	2	15052.7	3.36	0.1712
B(M)	3	4474.2	-	-
D	5	34113.9	27.75	0.0001*
MD	10	2149.4	1.75	0.0950
ϵ	51	1229.2		

Note: * Significant at α level of 0.05

2. Since curing condition does not interact with the maximum aggregate size, the effects caused by the six curing conditions can be assessed by comparing the means of splitting tensile strength for six curing conditions, averaged over the other factor.

The Duncan's multiple-range test at an α level of 0.05 was used to compare the means of splitting tensile strength. The results are summarized in Tables 9.23 and 9.24.

Table 9.23 shows the comparison of the effects of maximum aggregate size. It can be seen that the effects of maximum aggregate size on splitting tensile strength is statistically not significant.

Table 9.24 shows the comparison of the effects of curing condition. It shows that the 90-day and 28-day moist-curing conditions were not significantly different from one another, while they produced significantly higher splitting tensile strength than the other four curing conditions. The AC, CC, SC and 7-MC curing conditions exhibited about the same splitting tensile strength.

9.2.6 Results of Experimental Design No. 6 (Silica Fume Concrete)

The results of experimental design no. 6, which dealt with the effects of silica fume and curing condition on the splitting tensile strength are presented in this section. The results of tests on two batches of control concrete, as shown in Table C.3 in Appendix C, and eight batches of silica fume concrete, as shown in Tables C.10 and C.11, were analyzed in a manner similar to that done for design no. 5. Mix parameters and batch numbers for all ten batches are presented in Table 6.8.

Table 9.23 Grouping of Maximum Aggregate Size on the Basis of Splitting Tensile Strength from Duncan's Test (Design No. 5)

Maximum Aggregate Size (inches)	Means (psi)	Grouping
$\frac{3}{8}$	651	A
$\frac{3}{4}$	634	A
1	602	A

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

Table 9.24 Grouping of Curing Condition on the Basis of Splitting Tensile Strength from Duncan's Test (Design No. 5)

Curing Condition*	Means (psi)	Grouping
90-MC	710	A
28-MC	680	A
7-MC	613	B
SC	604	B
CC	589	B
AC	578	B

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

Plots of the mean splitting tensile strengths as functions of silica fume addition are shown in Figure 9.3. The maximum splitting tensile strength occurs at around 5% and 10% silica fume addition, for 28-day and 90-day moist curing, respectively.

ANOVA was performed on the test results by using a linear model as shown in Equation 6.6. Results of ANOVA are presented in Table 9.25. It can be seen that the effects of curing condition (D) and the interaction of silica fume addition and curing condition (SD) are significant, while the effect of percent addition of silica fume (S) is not significant. Therefore, the comparison of means must be made as follows:

1. Since percent addition of silica fume interacts with the curing condition, the effects of percent addition of silica fume can be evaluated by comparing the means of splitting tensile strength for five levels of percent addition of silica fume, averaged at each curing condition.
2. Since curing condition interacts with the percent addition of silica fume, the effects caused by the two curing conditions can be assessed by comparing the means of splitting tensile strength for two curing conditions, averaged at each percent addition of silica fume.

The Duncan's multiple-range test at an α level of 0.05 was used to compare the means of splitting tensile strength. Table 9.26 shows the comparison of the effects of percent addition of silica fume. It can be seen that the percent addition of silica fume had no significant effect on the splitting tensile strength for the 28-day moist curing. For the 90-day moist curing, the 10% addition of silica fume produced higher

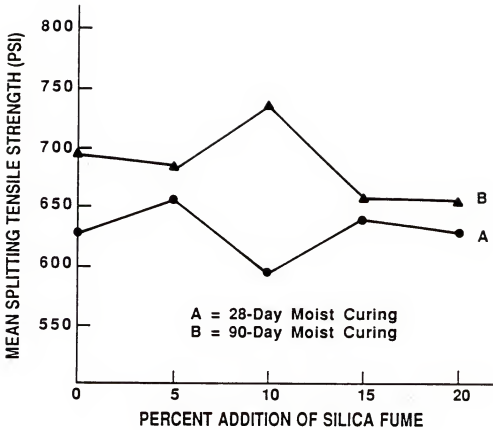


Figure 9.3 Effects of Silica Fume Addition on the Splitting Tensile Strength of Concrete with a Water-Cementitious Ratio of 0.33 and Brooksville Aggregate

Table 9.25 Result of ANOVA on Splitting Tensile Strength for Design No. 6

Factor	df	MS	F-Value	P
S	4	1177.4	0.80	0.5767
B(S)	5	1483.6	-	-
D	1	30747.0	31.32	0.0001*
SD	4	5494.3	5.60	0.0023*
ϵ	25	981.6		

Note: * Significant at α level of 0.05

Table 9.26 Grouping of Percent Addition of Silica Fume by Curing Condition on the Basis of Splitting Tensile Strength from Duncan's Test (Design No. 6)

Curing Condition*	Percent Addition of Silica Fume	Means (psi)	Grouping
28-MC	5	658	A
	15	640	A
	20	629	A
	0	628	A
	10	593	A
90-MC	10	736	A
	0	695	A B
	5	684	A B
	15	658	B
	20	653	B

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

splitting tensile strength than the other four levels of addition, but the differences between 0%, 5% and 10% additions are not significant. The 0%, 5%, 15% and 20% additions of silica fume produced splitting tensile strengths which were not significantly different from one another.

Table 9.27 shows the comparison of the effects of curing conditions. It shows that the 28-day and 90-day moist curing produced about the same splitting tensile strength for the 5%, 15%, and 20% additions of silica fume. For the 0% and 10% additions of silica fume, the 90-day moist curing produced significantly higher splitting tensile strength than the 28-day moist curing.

9.2.7 Results of Silica Fume Concrete Made with Calera Aggregate

This section presents the test results of silica fume concrete made with Calera aggregate. The results of tests on two batches of silica fume concrete are presented in Tables C.10 and C.11 in Appendix C. These results are compared with the results of tests on two batches of control concrete, which were made with Calera aggregate, cement Type II and a w/c-ratio of 0.33, and are presented in Table C.3 in Appendix C (batch numbers 13 and 40).

Figure 9.4 shows plots of the mean splitting tensile strengths as function of percent addition of silica fume for 28-day and 90-day moist curing. It can be seen that the 10% addition of silica fume produced higher splitting tensile strengths than the control concrete for the 28-day moist curing. For the 90-day moist curing, the splitting tensile strengths of silica fume concrete are lower than the control concrete. For the silica fume concrete, the 28-day moist curing produced higher splitting tensile strength than the 90-day moist curing. This discrepancy is believed to be due to the great scattering of the splitting

Table 9.27 Grouping of Curing Condition by Percent Addition of Silica Fume on the Basis of Splitting Tensile Strength from Duncan's Test (Design No. 6)

Percent Addition of Silica Fume	Curing Condition*	Means (psi)	Grouping
0	90-MC	695	A
	28-MC	629	B
5	90-MC	684	A
	28-MC	658	A
10	90-MC	736	A
	28-MC	593	B
15	90-MC	658	A
	28-MC	640	A
20	90-MC	653	A
	28-MC	629	A

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

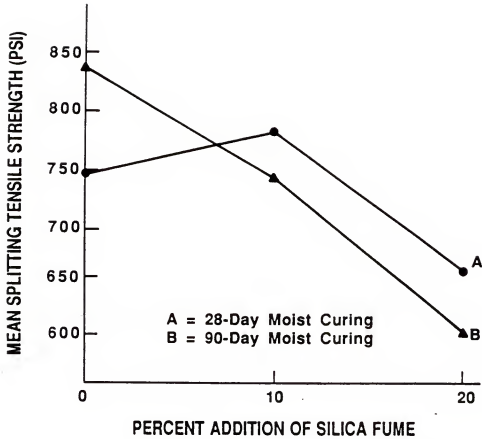


Figure 9.4 Effects of Silica Fume Addition on the Splitting Tensile Strength of Concrete with a Water-Cementitious Ratio of 0.33 and Calera Aggregate

tensile test results and the limited number of tests on this silica fume concrete. Further study and testing is needed to draw any definite conclusions with regards to the splitting tensile strength of silica fume concrete made with Calera aggregate.

9.3 Discussion of the Results

The results of a laboratory study on the splitting tensile strength of structural concrete have been presented in the previous sections. In this section, these results are discussed in accordance with the objectives of this study.

9.3.1 Effect of Aggregate Type

The effect of aggregate type on the splitting tensile strength is statistically significant at an α level of 0.05 (the probability of error of this statement is 0.05). Concretes using Calera aggregate exhibited higher splitting tensile strength than those using Brooksville or river gravel aggregate. The splitting tensile strengths of concretes using Brooksville aggregate are about the same as those using river gravel for the majority of the cases. These results are similar to those of the modulus of rupture and compressive strength tests. The properties of aggregate, especially its shape, surface texture and strength are believed to be the cause of difference of the effects of the different aggregate types.

9.3.2 Effect of Cement Type

The splitting tensile strengths of concretes using three different cement types are generally different from one another. The difference is significant at the early ages, especially at the 16-hour steam-curing and 7-day moist-curing conditions. Cement Type III produced higher

splitting tensile strength than the other two cement types for most of the cases. The fineness of cement is the main reason why cement Type III exhibited higher strength than cement Type I or II at the early ages. At 90-day moist curing, cement type had no effect on the splitting tensile strength. The degree of hydration for all cement types is believed to be about the same at the 90-day moist-curing condition.

Cement Type II produced lower splitting tensile strengths than cement Type I at the early ages. This is because cement Type II has a lower percentage of tricalcium silicate (C_3S) and fineness than cement Type I. The effect of cement type on the splitting tensile strength is also similar to the modulus of rupture and compressive strength.

9.3.3 Effect of W/C-Ratio

The w/c-ratio significantly affects the splitting tensile strength of concrete in this study. Concrete with a higher w/c-ratio exhibited lower splitting tensile strength. This is, because concrete with a higher w/c-ratio tends to have a higher volume of voids in the cement paste than that with a lower w/c-ratio. The w/c-ratio of 0.33 exhibited higher splitting tensile strengths than the w/c-ratios of 0.38 and 0.45 by about 8% to 10%. The general trend of the effect of w/c-ratio on the splitting tensile strength is also similar to that for the modulus of rupture and the compressive strength.

9.3.4 Effect of Maximum Aggregate Size

The ANOVA results show that the effect of the maximum aggregate size on the splitting tensile strength is insignificant at an α level of 0.05. The maximum aggregate size of $3/8$ inch exhibited slightly higher splitting tensile strength than the maximum aggregate sizes of $3/4$ inch and 1 inch, but the splitting tensile strengths of these three maximum

aggregate sizes are not significantly different from one another. The use of smaller maximum aggregate size increases the surface area of the aggregate and, thus, the bond strength between the aggregate and cement pastes is increased. This explains why the use of smaller maximum aggregate size produced higher splitting tensile strength than the larger maximum aggregate sizes. The effect of the maximum aggregate size on the splitting tensile strength is also similar to that for the modulus of rupture and compressive strength.

9.3.5 Effect of Fly Ash

The 20% fly ash replacement of cement does not have any significant effect on the splitting tensile strength of the concrete tested. This can be clearly seen from Table 9.16 and Figures 9.1 and 9.2. The splitting tensile strength of the fly ash concrete is about the same as that of the control concrete for all cases. It can be noted that the replacement of 20% of cement by fly ash does not affect the tensile strength, while it affects the compressive strength of concrete.

9.3.6 Effect of Silica Fume

9.3.6.1 Silica fume concrete made with Brooksville aggregate. The effect of percent addition of silica fume on the splitting tensile strength is insignificant while the interaction of silica fume and curing condition is significant. At 90-day moist curing, the maximum splitting tensile strength occurs at around 10% addition of silica fume, and it is about 6% higher than that of the control concrete. The 5%, 15%, and 20% additions of silica fume exhibited lower splitting tensile strength than the control concrete by about 2% to 6%. At 28-day moist curing, the 5% addition of silica fume exhibited the highest splitting tensile strength. The 10% addition of silica fume produced lower

splitting tensile strength than the control concrete by about 5%. These observations can be clearly seen in Figure 9.3 and Table 9.26. The maximum splitting tensile strength occurs at a different percentage of addition of silica fume for the two curing conditions. This is believed to be due to the great variability of the splitting tensile strength test results.

The addition of silica fume contributed less benefit to the splitting tensile strength than to the modulus of rupture and compressive strength. The silica fume concrete used in this study reaches its maximum splitting tensile strength at a silica fume addition of 10% by weight of cement at 90-day moist curing. This is in agreement with the case of compressive strength. However, at 28-day moist curing, the 5% addition of silica fume produced the highest splitting tensile strength. This result is not in agreement with both modulus of rupture and compressive strength.

9.3.6.2 Silica fume concrete made with Calera aggregate. The results of the splitting tensile strength test on silica fume concrete made with Calera aggregate are not in agreement with those of the silica fume concrete made with Brooksville aggregate at both curing conditions. The great variability of the splitting tensile strength test results is also believed to be the main reason. Therefore, further study and testing is needed to investigate the splitting tensile strength of silica fume concrete made with Calera aggregate before any definite conclusions can be made.

9.3.7 Effect of Curing Condition

The curing condition is one of the major factors affecting the splitting tensile strength. This can be clearly seen from all the ANOVA

results in this chapter. The 90-day moist-curing condition exhibited the highest splitting tensile strength. The 28-MC curing condition produced higher splitting tensile strength than the AC, CC and SC curing conditions. The splitting tensile strengths of the air-cured specimens and specimens with curing compound are not significantly different from each other at an α level of 0.05. The 7-MC and SC curing conditions produced about the same splitting tensile strength. The effect of curing condition on the splitting tensile strength is similar to that on the modulus of rupture. Therefore, the discussion in the previous chapter on the effect of curing condition on modulus of rupture also applies to the effect of curing condition on splitting tensile strength.

For fly ash concrete, the splitting tensile strengths of the steam-cured specimens are 72%, 78% and 91% of those of the 28-day moist-cured specimens for water-cementitious ratios of 0.45, 0.38 and 0.33, respectively. This again indicates that the effectiveness of steam curing on fly ash concrete increases as the water-cementitious ratio decreases.

9.4 Summary of Results

The findings presented in this chapter on the effects of mix parameters and curing conditions on the splitting tensile strength are summarized as follows:

1. The w/c-ratio is the major parameter affecting the splitting tensile strength. Generally, the higher the w/c-ratio, the lower the splitting tensile strength would be.
2. Curing condition is also one of the major parameters affecting the splitting tensile strength. The continued moist curing tends to increase the splitting tensile strength. The 90-day

moist curing produces the highest splitting tensile strength. The 28-MC curing condition exhibits higher splitting tensile strength than the AC, CC and SC curing conditions. The splitting tensile strengths of the AC and CC curing conditions are not significantly different from each other at an α level of 0.05. Also, the 7-MC and SC curing conditions produce approximately the same splitting tensile strength.

3. The aggregate type has some effect on the splitting tensile strength. Concretes made with Calera aggregate have higher splitting tensile strengths than those made with Brooksville or river gravel aggregate. The splitting tensile strengths of concretes using Brooksville aggregate are about the same as those using river gravel for most of the cases.
4. Cement type also has some effects on the splitting tensile strength, especially at the early ages. Concretes using Type III cement produce higher splitting tensile strengths than those using Type I or II cement for the majority of the cases. Cement Type II generally produced lower splitting tensile strength than cement Type I. However, in some cases, the splitting tensile strengths of these two types of cement are not significantly different at an α level of 0.05. At 90-day moist curing, the cement type has no significant effect on the splitting tensile strength.
5. The splitting tensile strengths of the maximum aggregate sizes of $\frac{3}{8}$ inch, $\frac{3}{4}$ inch and 1 inch are not significantly different from one another at an α level of 0.05.

6. The splitting tensile strengths of fly ash and control concrete are about the same for all cases. Therefore, the replacement of 20% of cement by fly ash has no significant effect on the splitting tensile strength.
7. The effects of w/c-ratio, curing condition, aggregate type, cement type, maximum aggregate size and the 20% replacement of cement by fly ash on the splitting tensile strength are similar to their effects on the modulus of rupture.
8. The silica fume concrete made with Brooksville aggregate produces the highest splitting tensile strength at a silica fume addition of 5% and 10% by weight of cement at the 28-day and 90-day moist curing, respectively.

CHAPTER 10 RESULTS OF PERMEABILITY TESTS

10.1 Introduction

The permeability of concrete plays a very important role in influencing the durability of a concrete structure. It controls the rate of flow of water and chemicals such as chlorides and sulfates into the concrete affecting the durability of the concrete structure. Therefore, this property of concrete is of interest to nearly all designers of concrete structures [1]. Test data on this property were limited, especially for structural concretes used in Florida. This testing program was conducted to determine the magnitudes of coefficients of permeability of structural concretes used in Florida, and to study the effects of various mix parameters and curing conditions on them.

This chapter presents the results of permeability tests in all the six experimental designs. The details of all the experimental designs are presented in Chapter 6. The general laboratory procedure described in Chapters 3 and 4 was used to prepare and test the concrete specimens. The method of analysis of the test results are also described in Chapter 6.

10.2 Analysis of Results

10.2.1 Results of Experimental Design No. 1

The results of experimental design no. 1, which dealt with the effects of aggregate type, cement type and curing condition on the

permeability are presented in this section. Results of tests on 18 batches, as shown in Table C.4 in Appendix C, were analyzed with the aid of the ANOVA statistical method. Mix parameters and batch numbers for all 18 batches are presented in Table 6.1. Table 10.1 presents the mean coefficients of permeability of concretes made with three different aggregates and three cement types at five curing conditions.

ANOVA was performed on the permeability data by using the linear model as shown in Equation 6.1. Results of ANOVA are summarized in Table 10.2. It can be seen that the effects of aggregate type (A), cement type (C), curing condition (D) and the interaction ACD are significant while the interactions AC, AD and CD are insignificant.

Since the three-factor interaction (ACD) is significant, the comparison of means must be done as follows:

1. Comparison of aggregate type must be made at each cement type and curing condition.
2. Comparison of cement type must be made at each aggregate type and curing condition.
3. Comparison of curing condition must be made at each aggregate type and cement type.

The Duncan's multiple-range test at an α level of 0.05 was used to compare the means of coefficient of permeability. The results are summarized in Tables 10.3 through 10.5. Table 10.3 shows the comparison of the effects of aggregate types for various combinations of cement types and curing conditions. It can be seen that for cement Type III, all three aggregates produce permeabilities which are not significantly different from one another at all five curing conditions. For cement Type II, the effects of the three aggregate types are not significantly

Table 10.1 Mean Coefficients of Permeability of Concrete in Design No. 1

Aggregate Type	Cement Type	Coefficient of Permeability ($\times 10^{-12}$ inches/sec)				
		Curing Condition*				
		1	3	4	5	6
Brooksville	I	7.26	3.65	5.12	5.57	3.16
	II	4.79	3.85	5.23	4.98	3.09
	III	3.20	2.60	3.17	3.42	1.59
Calera	I	3.83	1.84	2.56	2.56	1.52
	II	2.86	2.14	3.16	3.51	1.73
	III	2.40	1.90	2.33	2.46	1.18
River Gravel	I	3.79	2.57	3.40	3.06	2.05
	II	6.29	2.79	3.72	4.37	2.54
	III	3.80	2.0	2.70	2.75	1.52

Note: * Curing conditions:

1 = 16 hours steam curing

2 = 7 days moist curing

3 = 28 days moist curing

4 = 3 days moist curing and 25 days in air

5 = 1 day in form and 27 days with curing compound

6 = 90 days moist curing

Table 10.2 Result of ANOVA on Coefficient of Permeability for Design No. 1

Factor	df	MS ($\times 10^{-24}$)	F-Value	P
A	2	40.85	8.11	0.0097*
C	2	24.85	4.93	0.0358*
AC	4	6.17	1.22	0.3659
B(AC)	9	5.04	-	-
D	4	27.68	54.70	0.0001*
AD	8	0.98	1.94	0.0833
CD	8	0.88	1.74	0.1232
ACD	16	1.25	2.46	0.0124*
BD(AC)	36	0.51	-	-
ϵ	90	0.12		

Note: * Significant at α level of 0.05

Table 10.3 Grouping of Aggregate Type by Cement Type and Curing Condition on the Basis of Coefficient of Permeability from Duncan's Test (Design No. 1)

Cement Type	Aggregate Type	Duncan's Grouping for Each Curing Condition				
		AC*	CC	SC	28-MC	90-MC
I	Brooksville	A (5.12)	A (5.57)	A (7.25)	A (3.65)	A (3.15)
	Calera	B (2.55)	B (2.55)	B (3.83)	B (1.84)	B (1.52)
	River Gravel	B (3.40)	B (3.05)	B (3.79)	AB (2.57)	AB (2.04)
II	Brooksville	A (5.23)	A (4.98)	B (4.79)	A (3.85)	A (3.09)
	Calera	B (3.16)	A (3.50)	C (2.86)	B (2.13)	A (1.72)
	River Gravel	B (3.72)	A (4.37)	A (6.28)	AB (2.79)	A (2.53)
III	Brooksville	A (3.17)	A (3.41)	A (3.20)	A (2.59)	A (1.59)
	Calera	A (2.33)	A (2.46)	A (2.39)	A (1.89)	A (1.17)
	River Gravel	A (2.69)	A (2.75)	A (3.80)	A (1.99)	A (1.51)

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

** Mean of coefficient of permeability ($\times 10^{-12}$ inches/sec)

Table 10.4 Grouping of Cement Type by Aggregate Type and Curing Condition on the Basis of Coefficient of Permeability from Duncan's Test

Aggregate Type	Cement Type	Duncan's Grouping for Each Curing Condition									
		AC*	CC	SC	28-MC	90-MC					
Brooksville	I	A	(5.12)	A	(5.57)	A	(7.25)	A	(3.65)	A	(3.15)
	II	A	(5.23)	A	(4.98)	B	(4.79)	A	(3.85)	A	(3.09)
	III	B	(3.17)	B	(3.41)	C	(3.20)	A	(2.59)	B	(1.59)
Calera	I	A	(2.55)	A	(2.55)	A	(3.83)	A	(1.84)	A	(1.52)
	II	A	(3.16)	A	(3.50)	A	(2.86)	A	(2.13)	A	(1.72)
	III	A	(2.33)	A	(2.46)	A	(2.39)	A	(1.89)	A	(1.17)
River Gravel	I	A	(3.4)	AB	(3.05)	B	(3.79)	A	(2.57)	A	(2.04)
	II	A	(3.72)	A	(4.37)	A	(6.28)	A	(2.79)	A	(2.53)
	III	A	(2.69)	B	(2.75)	B	(3.80)	A	(1.99)	A	(1.51)

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

** Mean of coefficient of permeability ($\times 10^{-12}$ inches/sec)

Table 10.5 Grouping of Curing Condition by Aggregate Type and Cement Type on the Basis of Coefficient of Permeability from Duncan's Test

Aggregate Type	Curing Condition*	Duncan's Grouping for Each Cement Type					
		I		II		III	
						**	
Brooksville	90-MC	C	(3.15)	B	(3.09)	B	(1.59)
	28-MC	C	(3.65)	AB	(3.85)	AB	(2.59)
	AC	B	(5.12)	A	(5.23)	A	(3.17)
	CC	B	(5.57)	A	(4.98)	A	(3.41)
	SC	A	(7.25)	A	(4.79)	A	(3.20)
Calera	90-MC	B	(1.52)	B	(1.72)	A	(1.17)
	28-MC	B	(1.84)	AB	(2.13)	A	(1.89)
	AC	AB	(2.55)	AB	(3.16)	A	(2.33)
	CC	AB	(2.55)	AB	(3.50)	A	(2.46)
	SC	A	(3.83)	A	(2.86)	A	(2.39)
River Gravel	90-MC	B	(2.04)	C	(2.53)	B	(1.51)
	28-MC	AB	(2.57)	C	(2.79)	B	(1.99)
	AC	AB	(3.40)	BC	(3.72)	AB	(2.69)
	CC	AB	(3.05)	B	(4.37)	AB	(2.75)
	SC	A	(3.79)	A	(6.28)	A	(3.80)

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

** Mean of coefficient of permeability ($\times 10^{-12}$ inches/sec)

different from one another at CC and 90-MC curing conditions. For the other combinations of cement types and curing conditions, the Calera aggregate generally produces a significantly lower permeability than the Brooksville aggregate, while the river gravel generally is not significantly different from the Calera aggregate.

Table 10.4 shows the comparison of the effects of cement types for various combinations of aggregate type and curing conditions. For concretes made with Brooksville aggregate, cement Type III generally produced lower permeability than cement Types I and II. For concretes using Calera aggregate, all three cement types exhibited about the same permeability at all five curing conditions. For concretes made with river gravel, cement Type II produced slightly higher permeability than cement Types I and III, while cement Type I produced the same permeability as cement Type III for all curing conditions.

Table 10.5 shows the comparison of the effects of curing conditions for various combinations of aggregate types and cement types. It can be seen that the 90-day moist-cured specimens gave the lowest permeability, but it is not significantly different from the 28-day moist curing. The 28-MC curing condition gave lower permeability than the AC, CC and SC curing conditions for all cases, but in some cases the permeabilities of these four curing conditions are not significantly different from one another. The AC curing condition exhibited the same permeability as the CC curing condition. The SC curing condition gave higher or the same permeability as the AC and CC curing conditions.

10.2.2 Results of Experimental Design No. 2

The results of experimental design no. 2, which dealt with the effects of aggregate type, w/c-ratio and curing condition on the

permeability are presented in this section. The results of tests on 18 batches, as shown in Table C.4 in Appendix C, were analyzed in a manner similar to that done for the previous design. Mix parameters and batch numbers for all 18 batches are presented in Table 6.3. The mean coefficients of permeability of concretes in this design using three different aggregate types, three levels of w/c-ratio and five curing conditions are presented in Table 10.6.

ANOVA was performed on the permeability data by using the linear model as shown by Equation 6.2. Results of ANOVA are summarized in Table 10.7. It can be seen that all main effects and all interaction effects are statistically significant. It, therefore, follows that:

1. Comparison of aggregate type must be made at each w/c-ratio, for each curing condition.
2. Comparison of w/c-ratio must be made at each aggregate type and curing condition.
3. Comparison of curing condition must be made at each aggregate type and w/c-ratio.

The Duncan's multiple-range test at an α level of 0.05 was used to compare the means of coefficient of permeability. Table 10.8 shows the comparison of the effects of aggregate types for various combinations of w/c-ratios and curing conditions. It can be seen that at a w/c-ratio of 0.33, the effects of the three aggregate types are not significantly different from one another at the curing conditions of 28-MC and 90-MC. At other combinations of w/c-ratios and curing conditions, concretes made with Calera aggregate have significantly lower permeability than those made with the Brooksville aggregate. For w/c-ratios of 0.38 and 0.45, the permeability of concretes using Brooksville aggregate is

Table 10.6 Mean Coefficients of Permeability of Concrete in Design No. 2

Aggregate Type	W/C-Ratio	Coefficient of Permeability ($\times 10^{-12}$ inches/sec)				
		Curing Condition*				
		1	3	4	5	6
Brooksville	0.45	13.61	5.11	5.93	5.96	5.08
	0.38	8.0	5.16	6.75	6.76	5.07
	0.33	7.26	3.65	5.12	5.57	3.16
Calera	0.45	5.12	3.04	3.44	3.94	2.58
	0.38	2.60	2.19	2.72	2.76	1.91
	0.33	3.83	1.84	2.56	2.56	1.52
River Gravel	0.45	15.29	6.11	7.04	8.16	5.43
	0.38	6.79	4.11	5.17	5.38	3.70
	0.33	3.79	2.57	3.40	3.06	2.05

Note: * See footnotes on Table 10.1

Table 10.7 Result of ANOVA on Coefficient of Permeability for Design No. 2

Factor	df	MS ($\times 10^{-24}$)	F-Value	P
A	2	183.15	32.22	0.0001*
W	2	130.63	22.98	0.0003*
AW	4	26.82	4.72	0.025*
B(AW)	9	5.69	-	-
D	4	87.21	107.76	0.0001*
AD	8	7.16	8.85	0.0001*
WD	8	16.52	20.41	0.0001*
AWD	16	3.50	4.32	0.0001*
BD(AW)	36	0.81	-	-
ϵ	90	0.22		

Note: * Significant at α level of 0.05

Table 10.8 Grouping of Aggregate Type by W/C-Ratio and Curing Condition on the Basis of Coefficient of Permeability from Duncan's Test (Design No. 2)

W/C-Ratio	Aggregate Type	Duncan's Grouping for Each Curing Condition				
		AC*	CC	SC	28-MC	90-MC
0.33	Brooksville	A (5.12)	A (5.57)	A (7.25)	A (3.65)	A (3.15)
	Calera	B (2.55)	B (2.55)	B (3.83)	A (1.84)	A (1.52)
	River Gravel	AB (3.40)	B (3.05)	B (3.79)	A (2.57)	A (2.04)
0.38	Brooksville	A (6.75)	A (6.75)	A (7.99)	A (5.16)	A (5.07)
	Calera	B (2.72)	B (2.75)	B (2.60)	B (2.18)	B (1.91)
	River Gravel	A (5.17)	A (5.38)	A (6.79)	A (4.11)	AB (3.70)
0.45	Brooksville	A (5.92)	B (5.96)	A (13.61)	A (5.11)	A (5.08)
	Calera	B (3.44)	C (3.93)	B (5.12)	B (3.03)	B (2.57)
	River Gravel	A (7.04)	A (8.16)	A (15.29)	A (6.11)	A (5.08)

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

** Mean of coefficient of permeability ($\times 10^{-12}$ inches/sec)

not significantly different from those using river gravel for all curing conditions, except for the combination of w/c-ratio of 0.45 and CC curing condition, where the river gravel produces a higher permeability.

Table 10.9 displays the comparison of the effects of w/c-ratios for various combinations of aggregate types and curing conditions. It can be seen that the w/c-ratio of 0.33 shows lower permeability than the w/c-ratios of 0.38 and 0.45 for most of the cases, but in some cases the differences are not significant at an α level of 0.05. For concretes using Brooksville and Calera aggregates, the differences in the effects of w/c-ratios are not significant at the AC, CC, 28-MC and 90-MC curing conditions. For concretes using river gravel, the effects of w/c-ratio on the permeability is more than that for concretes using the other two aggregates.

Table 10.10 presents the comparison of the effects of curing conditions for various combinations of aggregate types and w/c-ratios. It can be seen that the 90-MC curing condition produced about the same permeability as the 28-MC curing condition. The 28-day moist curing produced lower permeability than the AC and CC curing conditions, but the permeabilities produced by these three curing conditions are not significantly different from one another for most of the cases. The AC and CC curing conditions produced approximately the same permeability for all cases. The SC curing condition gave higher permeability than the other four curing conditions for all the cases, but in some cases the differences are statistically not significant.

10.2.3 Results of Experimental Design No. 3

This section presents the results of experimental design no. 3, which dealt with the effects of w/c-ratio, cement type and curing

Table 10.9 Grouping of W/C-Ratio by Aggregate Type and Curing Condition on the Basis of Coefficient of Permeability from Duncan's Test (Design No. 2)

Aggregate Type	W/C-Ratio	Duncan's Grouping for Each Curing Condition				
		AC*	CC	SC	28-MC	90-MC
Brooksville	0.33	A (5.12)	A (5.57)	B (7.25)	A (3.65)	B (3.15)
	0.38	A (6.75)	A (6.75)	B (7.99)	A (5.16)	A (5.07)
	0.45	A (5.92)	A (5.96)	A (13.61)	A (5.11)	A (5.08)
Calera	0.33	A (2.55)	A (2.55)	AB (3.83)	A (1.84)	A (1.52)
	0.38	A (2.72)	A (2.75)	B (2.60)	A (2.18)	A (1.91)
	0.45	A (3.44)	A (3.93)	A (5.12)	A (3.03)	A (2.57)
River Gravel	0.33	B (3.40)	C (3.05)	C (3.79)	B (2.57)	B (2.04)
	0.38	B (5.17)	B (5.38)	B (6.79)	B (4.11)	AB (3.70)
	0.45	A (7.04)	A (8.16)	A (15.29)	A (6.11)	A (5.43)

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

** Mean of coefficient of permeability ($\times 10^{-12}$ inches/sec)

Table 10.10 Grouping of Curing Condition by Aggregate Type and W/C-Ratio on the Basis of Coefficient of Permeability from Duncan's Test (Design No. 2)

Aggregate Type	Curing Condition*	Duncan's Grouping for Each W/C-Ratio					
		0.33		0.38		0.45	
Brooksville		**					
	90-MC	D	(3.15)	B	(5.07)	B	(5.08)
	28-MC	CD	(3.65)	B	(5.16)	B	(5.11)
	AC	BC	(5.12)	AB	(6.75)	B	(5.92)
	CC	B	(5.57)	AB	(6.75)	B	(5.96)
	SC	A	(7.25)	A	(7.99)	A	(13.61)
Calera	90-MC	B	(1.52)	A	(1.91)	B	(2.57)
	28-MC	B	(1.84)	A	(2.18)	B	(3.03)
	AC	AB	(2.55)	A	(2.72)	AB	(3.44)
	CC	AB	(2.55)	A	(2.75)	AB	(3.93)
	SC	A	(3.83)	A	(2.60)	A	(5.12)
River Gravel	90-MC	A	(2.04)	B	(3.70)	C	(5.43)
	28-MC	A	(2.57)	B	(4.11)	C	(6.11)
	AC	A	(3.40)	AB	(5.17)	BC	(7.04)
	CC	A	(3.05)	AB	(5.38)	B	(8.16)
	SC	A	(3.79)	A	(6.79)	A	(15.29)

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

** Mean of coefficient of permeability ($\times 10^{-12}$ inches/sec)

condition on the permeability. The results of tests on 18 batches shown in Table C.4 in Appendix C were analyzed in a manner similar to that done for the previous two designs. Mix parameters and batch numbers for all 18 batches can be found in Table 6.4. Comparison of mean coefficients of permeability of concretes using three different levels of w/c-ratio and three cement types at the six curing conditions are presented in Table 10.11.

ANOVA was performed on the test results by using the linear model as shown by Equation 6.3. The results are summarized in Table 10.12. It can be seen that the effects of w/c-ratio (W), curing condition (D) and the interactions WC, WD and WCD are statistically significant while the effects of cement type (C) and the interaction CD are insignificant. It, therefore, follows that:

1. Comparison of w/c-ratio must be made at each cement type and curing condition.
2. Comparison of cement type must be made at each w/c-ratio and curing condition.
3. Comparison of curing condition must be made at each w/c-ratio and cement type.

The comparison of means was done by Duncan's test at an α level of 0.05. The results are summarized in Tables 10.13 through 10.15. Table 10.13 shows the comparison of the effects of w/c-ratios for various combinations of cement types and curing conditions. It can be seen that the w/c-ratio of 0.33 produced lower permeability than the w/c-ratios of 0.38 and 0.45, but the permeabilities corresponding to these three w/c-ratios are not significantly different from one another at α level of 0.05 for the curing conditions of AC, CC, 28-MC and 90-MC. For the

Table 10.11 Mean Coefficient of Permeability of Concrete in Design No. 3

W/C-Ratio	Cement Type	Coefficient of Permeability ($\times 10^{-12}$ inches/sec)				
		Curing Condition*				
		1	3	4	5	6
0.45	I	13.61	5.11	5.93	5.96	5.08
	II	25.0	7.15	7.79	7.57	6.36
	III	14.35	6.08	6.48	7.32	5.52
0.38	I	8.0	5.16	6.75	6.76	5.07
	II	8.67	5.19	5.84	6.35	4.27
	III	8.83	4.72	6.89	6.14	4.58
0.33	I	7.26	3.65	5.12	5.57	3.16
	II	4.79	3.85	5.23	4.98	3.09
	III	3.20	2.60	3.17	3.42	1.59

Note: * See footnotes on Table 10.1

Table 10.12 Result of ANOVA on Coefficient of Permeability for Design No. 3

Factor	df	MS ($\times 10^{-24}$)	F-Value	P
W	2	314.4	36.89	0.0001*
C	2	31.2	3.67	0.0687
WC	4	33.5	3.94	0.0409*
B(WC)	9	8.5	-	-
D	4	209.4	52.69	0.0001*
WD	8	69.9	17.58	0.0001*
CD	8	7.7	1.94	0.0832
WCD	16	10.1	2.55	0.0098*
BD(WC)	36	3.97	-	-
ϵ	90	0.31		

Note: * Significant at α level of 0.05

Table 10.13 Grouping of W/C-Ratio by Cement Type and Curing Condition on the Basis of Coefficient of Permeability from Duncan's Test (Design No. 3)

Cement Type	W/C-Ratio	Duncan's Grouping for Each Curing Condition				
		AC*	CC	SC	28-MC	90-MC
I	0.33	A (5.12)	A (5.57)	B (7.25)	A (3.65)	A (3.15)
	0.38	A (6.75)	A (6.75)	B (7.99)	A (5.16)	A (5.07)
	0.45	A (5.92)	A (5.96)	A (13.61)	A (5.11)	A (5.08)
II	0.33	A (5.23)	A (4.98)	B (4.79)	A (3.85)	A (3.09)
	0.38	A (5.84)	A (6.35)	B (8.67)	A (5.19)	A (4.26)
	0.45	A (7.79)	A (7.57)	A (24.99)	A (7.15)	A (6.35)
III	0.33	A (3.17)	A (3.41)	C (3.20)	A (2.59)	A (1.59)
	0.38	A (6.89)	A (6.13)	B (8.82)	A (4.72)	A (4.58)
	0.45	A (6.48)	A (7.32)	A (14.34)	A (6.08)	A (5.52)

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

** Mean of coefficient of permeability ($\times 10^{-12}$ inches/sec)

Table 10.14 Grouping of Cement Type by W/C-Ratio and Curing Condition on the Basis of Coefficient of Permeability from Duncan's Test (Design No. 3)

W/C-Ratio	Cement Type	Duncan's Grouping for Each Curing Condition				
		AC*	CC	SC	28-MC	90-MC
0.33	I	A (5.12)	A (5.57)	A (7.25)	A (3.65)	A (3.15)
	II	A (5.23)	A (4.98)	A (4.79)	A (3.85)	A (3.09)
	III	A (3.17)	A (3.41)	A (3.20)	A (2.59)	A (1.59)
0.38	I	A (6.75)	A (6.75)	A (7.99)	A (5.16)	A (5.07)
	II	A (5.84)	A (6.35)	A (8.67)	A (5.19)	A (4.26)
	III	A (6.89)	A (6.13)	A (8.82)	A (4.72)	A (4.58)
0.45	I	A (5.92)	A (5.96)	B (13.61)	A (5.11)	A (5.08)
	II	A (7.79)	A (7.57)	A (24.99)	A (7.15)	A (6.35)
	III	A (6.48)	A (7.32)	B (14.34)	A (6.08)	A (5.52)

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

** Mean of coefficient of permeability ($\times 10^{-12}$ inches/sec)

Table 10.15 Grouping of Curing Condition by W/C-Ratio and Cement Type on the Basis of Coefficient of Permeability from Duncan's Test (Design No. 3)

W/C-Ratio	Curing Condition*	Duncan's Grouping for Each Cement Type					
		I		II		III	
0.33		**					
	90-MC	A	(3.15)	A	(3.09)	A	(1.59)
	28-MC	A	(3.65)	A	(3.85)	A	(2.59)
	AC	A	(5.12)	A	(5.23)	A	(3.17)
	CC	A	(5.57)	A	(4.98)	A	(3.41)
	SC	A	(7.25)	A	(4.79)	A	(3.20)
0.38	90-MC	A	(5.07)	B	(4.26)	A	(4.58)
	28-MC	A	(5.16)	AB	(5.19)	A	(4.72)
	AC	A	(6.75)	AB	(5.84)	A	(6.89)
	CC	A	(6.75)	AB	(6.35)	A	(6.13)
	SC	A	(7.99)	A	(8.67)	A	(8.82)
0.45	90-MC	B	(5.08)	B	(6.35)	B	(5.52)
	28-MC	B	(5.11)	B	(7.15)	B	(6.08)
	AC	B	(5.92)	B	(7.79)	B	(6.48)
	CC	B	(5.96)	B	(7.57)	B	(7.32)
	SC	A	(13.61)	A	(24.99)	A	(14.34)

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

** Mean of coefficient of permeability ($\times 10^{-12}$ inches/sec)

steam-cured specimens, concretes with w/c-ratio of 0.45 produced significantly higher permeability than those with w/c-ratios of 0.38 and 0.33.

Table 10.14 displays the comparison of the effects of cement types for various combinations of w/c-ratios and curing conditions. It can be seen that the difference in the effects of all three types of cement are statistically not significant for all w/c-ratios and curing conditions, except for the w/c-ratio of 0.45 and SC curing condition. For the concrete with w/c-ratio of 0.45 and SC curing condition, cement Type II produced significantly higher permeability than cement Types I and III.

Table 10.15 presents the comparison of the effects of curing conditions for various combinations of w/c-ratios and cement types. It can be seen that the 90-day moist curing resulted in lower permeability than the AC, CC and 28-MC curing conditions, but the differences are not significant at a level of 0.05 for all cases. For the w/c-ratios of 0.45 the SC curing condition gave significantly higher permeability than the other four curing condition, but for the w/c-ratios of 0.38 and 0.33, the SC curing condition gave about the same permeability as the other four curing conditions, except for the w/c-ratio of 0.38 and cement Type II.

10.2.4 Results of Experimental Design No. 4 (Fly Ash Concrete)

The results of experimental design no. 4, which dealt with the effects of w/c-ratio, fly ash and curing condition on the permeability are presented in the section. The results of tests on 12 batches were analyzed in a manner similar to the previous three designs. Mix parameters and batch numbers for all 12 batches are presented in Table 6.5.

Comparisons of mean coefficients of permeability between the control concrete and fly concrete are presented in Table 10.16.

Figures 10.1 and 10.2 show plots of coefficients of permeability as functions of water-cementitious ratio for 28-day and 90-day moist curing, respectively. It can be clearly seen that the coefficient of permeability decreases as the water-cementitious ratio decreases. The fly ash concrete has a much lower permeability than the control concrete for these two curing conditions.

ANOVA was performed on the permeability data by using a linear model as shown in Equation 6.4. The results of ANOVA are summarized in Table 10.17. The ANOVA results show that only the interaction of fly ash and w/c-ratio (FW) are not significant, and all the main effects and the other interactions are significant. It, therefore, follows that:

1. Comparison of the effects of fly ash replacement must be made at each w/c-ratio and curing condition.
2. Comparison of effects of w/c-ratios must be made at each percent of fly ash and curing condition.
3. Comparison of the effects of curing conditions must be made at each w/c-ratio and percent of fly ash.

The Duncan's multiple-range test at an α level of 0.05 was used to compare the means of coefficient of permeability. Table 10.18 shows the comparison of the effects of fly ash for various combinations of w/c-ratios and curing conditions. It can be seen that the fly ash concrete showed lower permeability than the control concrete, but the difference is not significant for w/c-ratio of 0.33 at all curing conditions. For w/c-ratio of 0.38, the permeability of fly ash concrete is not significantly different from that of the control concrete at curing conditions

Table 10.16 Mean Coefficient of Permeability of Control Concrete and Fly Ash Concrete

Percent Fly Ash	W/(C+F) Ratio	Coefficient of Permeability ($\times 10^{-12}$ inches/sec)				
		Curing Condition*				
		1	3	4	5	6
0	0.45	24.99	7.15	7.79	7.57	6.35
	0.38	8.67	5.19	5.84	6.35	4.26
	0.33	4.79	3.85	5.23	4.98	3.09
20	0.45	14.19	4.20	10.31	12.54	2.40
	0.38	3.84	3.84	4.67	4.50	3.02
	0.33	3.81	2.74	3.77	3.67	1.77

Note: * See footnotes on Table 10.1

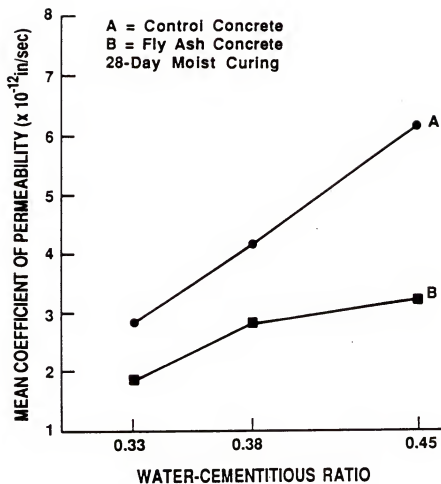


Figure 10.1 Effects of Water-Cementitious Ratio on the Coefficient of Permeability of Control and Fly Ash Concretes at 28-Day Moist Curing

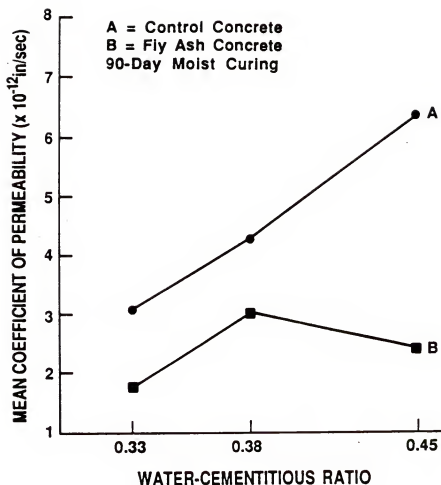


Figure 10.2 Effects of Water-Cementitious Ratio on the Coefficient of Permeability of Control and Fly Ash Concretes at 90-Day Moist Curing

Table 10.17 Result of ANOVA on Coefficient of Permeability for Design No. 4

Factor	df	MS ($\times 10^{-24}$)	F-Value	P
F	1	96.2	10.49	0.0177*
W	2	398.0	43.38	0.0003*
FW	2	2.3	0.25	0.7853
B(FW)	6	9.2	-	-
D	4	151.7	31.98	0.0001*
FD	4	34.4	7.26	0.0006*
WD	8	74.2	15.63	0.0001*
FWD	8	23.0	4.85	0.0012*
BD(FW)	24	4.7	-	-
ϵ	60	0.32		

Note: * Significant at α level of 0.05

Table 10.18 Grouping of Percent Replacement of Fly Ash by W/C-Ratio and Curing Condition on the Basis of Coefficient of Permeability from Duncan's Test (Design. No. 4)

W/C-Ratio	Percent Fly Ash	Duncan's Grouping for Each Curing Condition				
		AC*	CC	SC	28-MC	90-MC
0.33	0	A (5.23)	A (4.98)	A (4.79)	A (3.85)	A (3.09)
	20	A (3.77)	A (3.67)	A (3.81)	A (2.74)	A (1.77)
0.38	0	A (5.84)	A (6.35)	A (8.67)	A (5.19)	A (4.26)
	20	A (4.67)	A (4.50)	B (3.84)	A (3.84)	A (3.02)
0.45	0	A (7.79)	B (7.57)	A (24.99)	A (7.15)	A (6.35)
	20	A (10.31)	A (12.54)	B (14.19)	A (4.20)	B (2.40)

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

** Mean of coefficient of permeability ($\times 10^{-12}$ inches/sec)

of AC, CC, 28-MC and 90-MC, but at the SC curing condition, the permeability of fly ash concrete is significantly lower than that of the control concrete. For the air-curing and curing-compound conditions, the control concrete with a w/c-ratio of 0.45 has a lower permeability than the fly ash concrete of the same condition, but the difference is significant only for the curing compound condition. For the SC, 28-MC and 90-MC, the fly ash concrete with a water-cementitious ratio of 0.45 exhibited lower permeability than the control concrete, but for the 28-MC curing condition, the permeabilities of both concretes are not significantly different.

Table 10.19 displays the comparison of the effects of w/c-ratios for various combinations of percent of fly ash and curing conditions. It can be seen that the lower w/c-ratio produced lower permeability than the higher w/c-ratio for both concretes, but the differences are not significant in some cases. For the control concrete, the differences between the three w/c-ratios are not significant at the AC, CC and 90-MC curing conditions. For the fly ash concrete, the water-cementitious ratio of 0.45 produced significantly higher permeability than the other two w/c-ratios at the AC, CC and SC curing conditions, while the differences of the effects of the three water-cementitious ratios are not significant at the 28-MC and 90-MC curing conditions.

Table 10.20 shows the comparisons of the effects of curing conditions for various combinations of percent of fly ash and w/c-ratios. For the w/c-ratios of 0.33 and 0.38, the differences in the effects of curing conditions are not significant for both concretes. For the w/c-ratio of 0.45, the differences in the effects of curing conditions are relatively more significant. For fly ash concrete, the 90-MC and 28-MC

Table 10.19 Grouping of W/C-Ratio by Percent Replacement of Fly Ash and Curing Condition on the Basis of Coefficient of Permeability from Duncan's Test (Design No. 4)

Percent Fly Ash	W/C-Ratio	Duncan's Grouping for Each Curing Condition				
		AC*	CC	SC	28-MC	90-MC
0	0.33	A (5.23)	A (4.98)	B (4.79)	B (3.85)	A (3.09)
	0.38	A (5.84)	A (6.35)	B (8.67)	A (5.19)	A (4.26)
	0.45	A (7.79)	A (7.57)	A (24.99)	A (7.15)	A (6.35)
20	0.33	B (3.77)	B (3.67)	B (3.81)	A (2.74)	A (1.77)
	0.38	B (4.67)	B (4.50)	B (3.84)	A (3.84)	A (3.02)
	0.45	A (10.31)	A (12.54)	A (14.19)	A (4.20)	A (2.40)

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

** Mean of coefficient of permeability ($\times 10^{-12}$ inches/sec)

Table 10.20 Grouping of Curing Condition by Percent Replacement of Fly Ash and W/C-Ratio on the Basis of Coefficient Permeability from Duncan's Test (Design No. 4)

Percent of Fly Ash	Curing Condition*	Duncan's Grouping for Each W/C-Ratio					
		0.33		0.38		0.45	
		**					
0	90-MC	A	(3.09)	A	(4.26)	B	(6.35)
	28-MC	A	(3.85)	A	(5.19)	B	(7.15)
	AC	A	(5.23)	A	(5.84)	B	(7.79)
	CC	A	(4.98)	A	(6.35)	B	(7.57)
	SC	A	(4.79)	A	(8.67)	A	(24.99)
20	90-MC	A	(1.77)	A	(3.02)	B	(2.40)
	28-MC	A	(2.74)	A	(3.84)	B	(4.20)
	AC	A	(3.77)	A	(4.67)	A	(10.31)
	CC	A	(3.67)	A	(4.50)	A	(12.54)
	SC	A	(3.81)	A	(3.84)	A	(14.19)

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

** Mean of coefficient of permeability ($\times 10^{-12}$ inches/sec)

curing conditions produced about the same permeability and significantly lower permeability than the AC, CC and SC curing conditions.

10.2.5 Results of Experimental Design No. 5

This section presents the results of experimental design no. 5, which dealt with the effects of maximum aggregate size and curing condition on the permeability. The result of tests on six batches, as shown in Table C.4 in Appendix C, were used for analysis of this design. Mix parameters and batch numbers for all six batches are displayed in Table 6.6. Table 10.21 presents the mean coefficients of permeability of concretes using three different maximum aggregate sizes at five curing conditions. It can be seen that the use of smaller maximum aggregate size produced lower permeability than the use of larger maximum aggregate size.

ANOVA was performed on the permeability data by using the linear model as shown in Equation 6.5. Results of ANOVA are summarized in Table 10.22. It can be seen that the effects of curing condition (D) and the interaction of maximum aggregate size and curing condition (MD) are statistically significant while the effects of maximum aggregate size (M) is not significant. Therefore, the comparison of means must be made as follows:

1. Since maximum aggregate size interacts with the curing condition, the effects of maximum aggregate size can be evaluated by comparing the means of coefficient of permeability for three maximum aggregate sizes at each curing condition.
2. Since curing condition interacts with the maximum aggregate size, the effects caused by the curing conditions can be assessed by comparing the means of coefficient of permeability for five curing conditions at each maximum aggregate size.

Table 10.21 Mean Coefficients of Permeability of Concrete in Design No. 5

Maximum Aggregate Size (inches)	Coefficient of Permeability ($\times 10^{-12}$ inches/sec)				
	Curing Condition*				
	1	3	4	5	6
$3/8$	5.14	3.30	3.63	4.17	2.40
$3/4$	7.26	3.65	5.12	5.57	3.16
1	12.69	5.28	7.16	6.36	4.49

Note: * See footnotes on Table 10.1

Table 10.22 Result of ANOVA on Coefficient of Permeability for Design No. 5

Factor	df	MS ($\times 10^{-24}$)	F-Value	P
M	2	61.9	3.05	0.1890
B(M)	3	20.3	-	-
D	4	44.1	54.0	0.0001*
MD	8	6.3	7.71	0.0010*
BD(M)	12	0.82	-	-
ϵ	30	0.19		

Note: * Significant at α level of 0.05

A comparison of means for maximum aggregate size for each curing condition was performed by using Duncan's test at an α level of 0.05. The results are summarized in Table 10.23. It can be seen that the maximum aggregate size of 1 inch produced significantly higher permeability than the maximum aggregate sizes of $3/8$ inch and $3/4$ inch at the AC and SC curing conditions. The maximum aggregate sizes of 1 inch and $3/4$ inch exhibited about the same permeability at the CC, 28-MC and 90-MC curing conditions. The maximum aggregate sizes of $3/8$ inch and $3/4$ inch resulted in approximately the same permeability for all curing conditions, except for the SC curing conditions, where the maximum aggregate size of $3/8$ inch resulted in significantly lower permeability than the maximum aggregate size of $3/4$ inch.

Similarly, a comparison of means for curing condition for each maximum aggregate size was performed by using Duncan's test, at the same α level. These results are displayed in Table 10.24. It shows that the 90-day moist curing produced the lowest permeability, but it is not significantly different from the 28-day moist curing. The 28-MC, AC and CC curing conditions resulted in about the same permeability for all three maximum aggregate sizes. The SC curing condition generally produced higher permeability than the other four curing conditions.

10.2.6 Results of Experimental Design No. 6 (Silica Fume Concrete)

The results of experimental design no. 6, which dealt with the effects of silica fume and curing condition on the permeability are presented in this section. The results of tests on two batches of control concrete, as shown in Table C.4 in Appendix C, and eight batches of silica fume concrete, as shown in Tables C.10 and C.11, were analyzed in

Table 10.23 Grouping of Maximum Aggregate Size by Curing Condition on the Basis of Coefficient of Permeability from Duncan's Test (Design No. 5)

Curing Condition*	Maximum Aggregate Size (inches)	Means ($\times 10^{-12}$ inches/sec)	Grouping
AC	3/8	3.63	B
	3/4	5.12	B
	1	7.16	A
CC	3/8	4.17	B
	3/4	5.57	A B
	1	6.36	A
SC	3/8	5.14	C
	3/4	7.25	B
	1	12.69	A
28-MC	3/8	3.29	B
	3/4	3.65	A B
	1	5.28	A
90-MC	3/8	2.40	B
	3/4	3.15	A B
	1	4.49	A

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

Table 10.24 Grouping of Curing Condition by Maximum Aggregate Size on the Basis of Coefficient of Permeability from Duncan's Test (Design No. 5)

Maximum Aggregate Size (inches)	Curing Condition*	Means (x 10 ⁻¹² inches/sec)	Grouping
3/8	90-MC	2.40	B
	28-MC	3.29	A B
	AC	3.63	A B
	CC	4.17	A B
	SC	5.14	A
3/4	90-MC	3.15	C
	28-MC	3.65	B C
	AC	5.12	B C
	CC	5.57	A B
	SC	7.25	A
1	90-MC	4.49	C
	28-MC	5.28	B C
	AC	7.16	B
	CC	6.36	B C
	SC	12.69	A

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

* See footnotes on Table 7.3

a manner similar to that done for design no. 5. Mix parameters and batch numbers for all ten batches are presented in Table 6.8.

Figure 10.3 shows plots of the mean coefficients of permeability as functions of percent addition of silica fume for 28-day and 90-day moist-curing conditions. It can be seen that the coefficient of permeability decreases as the silica fume addition increases to 10% and then seems to level off beyond 10%. The silica fume addition reduces the permeability of the control concrete by about 50% to 60% at 28-day moist curing and by about 35% to 55% at 90-day moist curing.

ANOVA was performed on the test results by using the linear model as shown in Equation 6.6. Results of ANOVA on the coefficients of permeability of silica fume are summarized in Table 10.25. It is seen that the effects of silica fume addition (S) is significant while the other factors are not significant.

Since the interaction between percent addition of silica fume and curing condition is not significant, the comparison of means of one main factor can be made, averaged over the other main factor.

The comparison of mean for percent addition of silica fume averaged over two curing conditions was performed by using Duncan's test at an α level of 0.05. The results are summarized in Table 10.26. It can be seen that all percent additions of silica fume produced approximately the same permeability. The permeability of control concrete is significantly higher than the silica fume concrete with 10%, 15% and 20% additions of silica fume.

Although the coefficients of permeability of 90-day moist curing are slightly lower than those at 28-day moist curing, the differences are statistically insignificant at a confidence level of 95%.

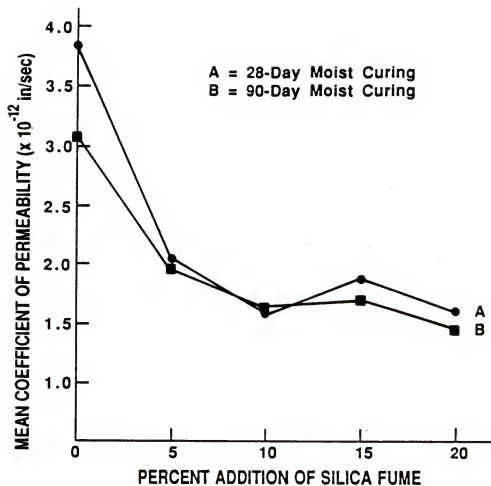


Figure 10.3 Effects of Silica Fume Addition on the Coefficient of Permeability of Concrete with a Water-Cementitious Ratio of 0.33 and Brooksville Aggregate

Table 10.25 Result of ANOVA of Coefficient of Permeability for Design No. 6

Factor	df	MS ($\times 10^{-24}$)	F-Value	P
S	4	5.1	7.73	0.0233*
B(S)	5	0.66	-	-
D	1	0.51	2.82	0.1541
SD	4	0.20	1.08	0.4569
BD(S)	5	0.18	-	-
ϵ	20	0.038		

Note: * Significant at α level of 0.05

Table 10.26 Grouping of Percent Addition of Silica Fume on the Basis of Coefficient of Permeability from Duncan's Test (Design No. 6)

Percent of Silica Fume	Means ($\times 10^{-12}$ inches/sec)	Grouping
0	3.47	A
5	2.02	A B
15	1.79	B
10	1.62	B
20	1.53	B

Note: - Means with the same letter are not significantly different ($\alpha = 0.05$)

10.2.7 Results of Silica Fume Concrete Made with Calera Aggregate

The results of permeability tests of silica fume concretes made with Calera aggregate are presented in this section. The test results of two batches of silica fume concretes made with Calera aggregate are presented in Tables C.10 and C.11 while the test results of control concrete are presented in Table C.4 in Appendix C (see batch numbers 13 and 40).

Plots of the mean coefficients of permeability as functions of percent addition of silica fume for 28-day and 90-day moist-curing conditions are shown in Figure 10.4. It can be seen that the coefficients of permeability of control and silica fume concrete are approximately the same at 28-day moist curing. At 90-day moist curing, the 10% addition produced lower permeability than the control concrete, while the 20% addition produced higher permeability than the control concrete.

10.3 Effects of Degree of Saturation on Permeability Test Results

A question arose from many researchers who are interested in the permeability test method developed for this research program, as to the effects of different degrees of saturation of test specimens on the results of the permeability test. A small experimental study was conducted in order to investigate the extent of the effects of the degree of saturation on the permeability test results. The effects of two degrees of saturation were investigated in this set of experiments.

Two concrete specimens, which were made from the same batch of concrete and subjected to the same curing condition were prepared for the permeability test as described in Chapter 3. One specimen was submerged in water for about 24 hours prior to test. This specimen was considered

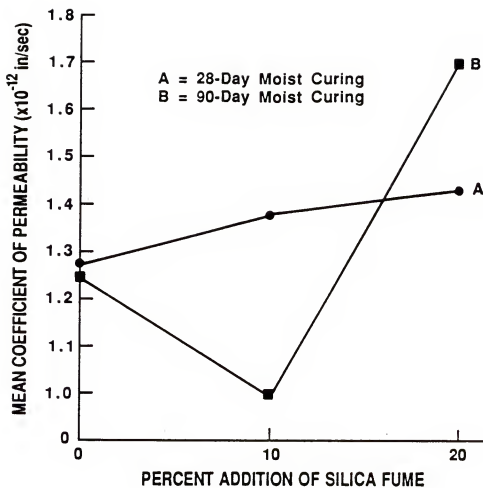


Figure 10.4 Effects of Silica Fume Addition on the Coefficient of Permeability of Concrete with a Water-Cementitious Ratio of 0.33 and Calera Aggregate

as partially saturated. The other specimen was saturated in water under vacuum. The vacuum saturation procedure similar to AASHTO T 277 [15] was performed. Figure 10.5 shows the vacuum saturation apparatus that was used for this study. The specimen was placed in a vacuum desiccator, then vacuum was applied to the dry specimen for three hours. De-aerated water was then drawn into the vacuum desiccator until it covered the specimen after which the vacuum pump was kept running. One hour later, the vacuum pump was turned off, and the specimen was soaked in the same water for an additional 20 hours. Then, these two specimens were ready to be tested in the permeability apparatus. Twelve pairs of specimens were prepared and tested in this manner. Table 10.27 presents the results of tests and the percent difference for all twelve pairs of specimens. It can be seen that the coefficients of permeability of vacuum-saturated specimens are higher than those of partially saturated specimens in the range of 0% to 30%, except for batch 43. The entrapped air in the concrete pores is believed to be the main factor for the difference in the test results, because the entrapped air can affect the flow path of water, which can result in reducing the rate of flow of water. The degree of saturation of the vacuum-saturated specimens is higher than the partially saturated specimens, and thus has less entrapped air in the concrete pores. This may explain why the coefficients of permeability of vacuum-saturated specimens are higher than the partially saturated specimens. The same phenomenon also happens in the case of soil. The coefficient of permeability of soil increases with an increase in the degree of saturation [55].

The average of the percent differences of coefficient of permeability is about 12%. Since all the permeability test specimens for this

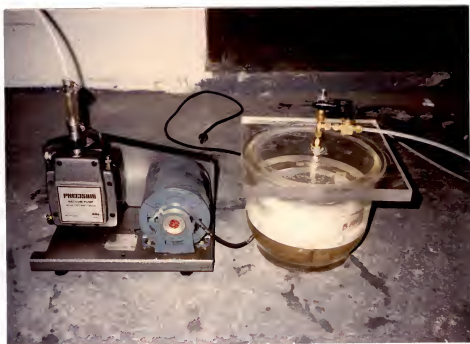


Figure 10.5 Vacuum Saturation Apparatus

Table 10.27 Comparison of Coefficients of Permeability of Partially and Vacuum-Saturated Specimens

Batch No.	Curing Condition*	Coefficient of Permeability**		Percent Difference
		K_1	K_2	
35	AC	3.43	4.25	23.91
36	28-MC	4.31	5.08	17.86
38	CC	2.32	3.09	33.19
41	28-MC	4.29	4.33	0.93
42	SC	8.11	9.48	16.89
42	28-MC	5.67	5.97	5.29
43	90-MC	4.30	4.25	-1.16
44	AC	6.37	6.79	6.19
45	28-MC	5.62	6.20	10.32
46	28-MC	2.95	3.14	6.44
47	28-MC	2.59	2.87	10.81
48	28-MC	4.32	4.53	4.86

Note: * See footnotes on Table 7.3

** Coefficient of permeability ($\times 10^{-12}$ inches/sec)

K_1 = for partially saturated specimens

K_2 = for vacuum-saturated specimens

study were soaked in water for about 24 hours prior to each test, an adjustment of an increase of 12% of the permeability test results can be used, if the coefficients of permeability at vacuum-saturated condition are needed. In the actual field condition, most of the structural concretes are in partially saturated condition. Thus, it is believed that the permeability test results from this study are more representative of the concrete in an actual field condition.

10.4 Discussion of the Results

The results of a laboratory study on concrete permeability have been presented in the previous sections. In this section, these results are discussed in accordance with the objectives of this study.

10.4.1 Effect of Aggregate Type

Aggregate type has some effects on the permeability of concrete. In design no. 1, for all concretes made with the same w/c-ratio (0.33), concretes using Calera aggregate generally resulted in lower permeability than those using Brooksville or river gravel aggregate. However, these three aggregate types produced approximately the same permeability for concrete using Type III cement.

Nyame [18] concluded that aggregate in concrete can affect the permeability in two opposing influences. The aggregate particles can obstruct the flow path resulting in reduced permeability, while cracks at the interface between hydrated cement and aggregate and the porosity of the aggregates can increase the permeability of the concrete. The cracks are caused by volume changes during setting and hardening [56].

Calera aggregate is a dense limestone and of a low permeability. Therefore, it can reduce the effective area over which flow can take

place. The bond between aggregate and cement paste is also a major factor affecting permeability. Concretes using river gravel exhibited higher permeability as compared to the Calera aggregate because of the lower bond strength of the river gravel. Concretes made with Brooksville aggregate produced higher permeability as compared with the Calera aggregate because of its porous nature. However, the differences in permeability of concretes using different aggregate types are not significant at an α level of 0.05 in some cases.

In design no. 2, concretes using Calera aggregate also exhibited lower permeability than those using Brooksville or river gravel aggregate for all cases. The reasons mentioned above are also applicable to this case. It can be noted that the differences in permeability of concretes using Brooksville and river gravel aggregates are dependent on the w/c-ratio. Brooksville aggregate produced higher permeability than river gravel by about 35% and 15%, for w/c-ratios of 0.33 and 0.38, respectively. For w/c-ratio of 0.45, Brooksville aggregate resulted in lower permeability than river gravel by about 15%, except for the 90-MC curing condition. This can be explained by pointing out the effect of w/c-ratio on the bond strength between the cement pastes and aggregate. At a lower w/c-ratio, the bond strength of the river gravel might be as high as that of the Brooksville aggregate. However, the Brooksville aggregate is more porous than the river gravel. This explains the relatively higher permeability of the concrete made with Brooksville aggregate. At the w/c-ratio of 0.45, the bond strength of Brooksville aggregate may be higher than that of the river gravel. For the concrete made with river gravel at w/c-ratio of 0.45, the water can flow through not only the pores in the cement pastes but also the interface between the

aggregate and the mortars. This is believed to be the main reason why the river gravel produced higher permeability than the Brooksville aggregate at the w/c-ratio of 0.45.

10.4.2 Effect of Cement Type

Powers et al. [14] found that at a given age and w/c-ratio, the permeability of cement pastes made with cements that hydrate rapidly is lower than those made with cements that hydrate slowly. At long-term curing, the cement type has no effect on the permeability.

The test results of this study showed that the cement type has minor effects on the permeability. In design no. 1, cement Type III resulted in lower permeability than cement Types I and II for all cases, but in some cases the differences are not significant at an α level of 0.05. This is because the fineness of cement Type III is higher than the other two cement types. The finer cement tends to produce paste that has lower porosity. At 28-day and 90-day moist curing, the permeabilities of these three cements are about the same. This can be explained by the fact that, at advanced age of concrete, the degree of hydration of these three cement types are approximately at close levels.

In design no. 2, all three cement types exhibited about the same permeability for all cases, except for concrete with w/c-ratio of 0.45 and SC curing condition. The results from these two designs, generally, are in agreement with the results by Powers.

10.4.3 Effect of W/C-Ratio

Many researchers have found that the w/c-ratio is one of the major parameters affecting the permeability of concrete [6,8,57]. The higher the w/c-ratio, the higher the permeability would be. This is due to the fact that the concrete with higher w/c-ratio would have a higher

volume of capillary pores in the cement paste than that with lower w/c-ratio.

In this study, the concretes with w/c-ratio of 0.33 produced the lowest permeability, concretes with w/c-ratio of 0.38 exhibited lower permeability than those with w/c-ratio of 0.45 for all three designs (design nos. 2 through 4), but the differences of permeability of these three w/c-ratios are not significant for some cases. The results from this study are in agreement with those by other researchers.

10.4.4 Effect of Maximum Aggregate Size

The maximum aggregate size has some effects on the permeability. The use of larger maximum aggregate size produced higher permeability than the use of smaller maximum aggregate size. Troxell et al. [10] stated that the larger maximum aggregate size, the higher the permeability would be. This is due to the development of large water voids on the underside of the coarse aggregate. The reduction of the specific area of the larger maximum aggregate size resulting in weaker bond with the cement paste is also believed to be another factor. Murata [13] and Ruettgers et al. [8] demonstrated that the smaller maximum aggregate size produced lower permeability than the larger maximum aggregate size. The results from this study are in agreement with the results from the researcher mentioned above.

10.4.5 Effect of Fly Ash

The use of fly ash as a cement replacement has significant effect on the permeability of concrete. From Table 10.16 and Figures 10.1 and 10.2, it can be seen that the fly ash concrete generally has a lower permeability than the control concrete, except for the air-curing and curing-compound conditions. At the air-curing and curing-compound

conditions, the fly ash concrete with a water-cementitious ratio of 0.45 has a higher permeability than the control concrete of the same condition. The differences in the permeabilities of control and fly ash concrete are not significant at an α level of 0.05 in some cases.

Hooton [26] stated that the use of fly ash can reduce the permeability. According to Hooton, this is because the pozzolanic action of fly ash reduces the size of coarser pores in cement pastes and reduces the risk of leaching calcium hydroxide. These reasons can also be used to explain the results of tests from this study. Marsh et al. [34] found that the pozzolanic reaction of fly ash in blended cement pastes can reduce the permeability as compared with the cement pastes at the same condition. Nagataki and Ujike [19] demonstrated that the fly ash concrete has lower air permeability than the control concrete at the 91-day moist curing. The results from this study are also in agreement with those from these researchers.

From Figures 10.1 and 10.2, it can be observed that the use of fly ash is more efficient in reducing permeability in concretes with higher w/c-ratios than those with lower w/c-ratios. This may be due to the fact that the control concrete with lower w/c-ratio (0.33) containing a smaller volume of voids in the cement pastes is a low permeability concrete. Therefore, the use of fly ash in this concrete will result in only slightly reduced permeability.

10.4.6 Effect of Silica Fume

10.4.6.1 Silica fume concrete made with Brooksville aggregate. The addition of silica fume to control concrete made with Brooksville aggregate resulted in a reduction of the coefficient of permeability of the control concrete by 50% to 60% at 28-day moist curing, and by 35% to 55%

at 90-day moist curing. The coefficients of permeability for concretes with 10% and 20% additions of silica fume are approximately equal and significantly lower than the coefficient of permeability of the control concrete. This is because the added silica fume reduces the volume of large pores and capillaries in the cement pastes [28].

Silica fume is known as the most effective material in reducing the permeability of concrete as compared to other pozzolanic materials. It is believed that the addition of silica fume would have a larger effect in reducing the coefficient of permeability of the control concrete than the results from this study indicated, if the silica fume were added to concretes that have higher w/c-ratios than 0.33. This is because concrete with a higher w/c-ratio will have a higher volume of voids than concrete with a lower w/c-ratio. The addition of silica fume to a concrete of high w/c-ratio will fill the coarser voids in the cement pastes, and thus reduce the permeability.

10.4.6.2 Silica fume concrete made with Calera aggregate. The coefficient of permeability of concrete with 10% addition of silica fume was lower than the control concrete by about 10% at 90-day moist curing. However, at 28-day moist curing, the coefficient of permeability of silica fume concrete was higher than the control concrete. This observation is not in agreement with the result on silica fume concrete made with Brooksville aggregate. Since this observation was based on a limited amount of test data, further testing is needed to substantiate this observation.

10.4.7 Effect of Curing Condition

Mcmillan and Lyse [6] found that continued moist curing is the most significant factor in reducing the permeability of concrete. In this

study, the 90-day moist-cured specimens exhibited the lowest coefficient of permeability as compared with the other four curing conditions. This is because the 90-day moist curing produces the highest degree of hydration, which results in the least volume of voids in the cement pastes.

The 28-MC curing condition produced lower concrete permeability than the AC and CC curing conditions. It is believed that the 28-day moist curing resulted in a higher degree of hydration than the air-curing and curing-compound conditions. Shrinkage may also be another factor that contributed to the increase in the permeability of the air-cured specimens and the specimens with curing compound. The steam-cured specimens produced the highest coefficient of permeability. This is due to the fact that these specimens had the lowest degree of hydration.

For fly ash concrete, it can be observed that proper curing is needed to produce a lower permeability concrete.

10.5 Summary of Results

The findings presented in this chapter on the effects of mix parameters and curing condition on the permeability of concrete are summarized as follows:

1. The w/c-ratio is one of the major parameters affecting the permeability of concrete. Generally, the lower the w/c-ratio, the lower the coefficient of permeability would be.
2. Curing condition is also one of the major parameters affecting the permeability of concrete. The continued curing tends to reduce the coefficient of permeability. The 90-MC curing condition gives the lowest coefficient of permeability. The 28-MC curing condition produces lower coefficients of permeability

than the AC, CC and SC curing conditions. The effects of AC and CC curing conditions on concrete permeability are not significantly different from each other. The steam-cured specimens exhibit the highest coefficient of permeability as compared with the specimens subjected to the other four curing conditions for the majority of cases.

3. The aggregate type has some effects on the coefficient of permeability. Concretes using Calera aggregate exhibit lower coefficient of permeability than those using Brooksville or river gravel aggregate. Concretes using Brooksville aggregate show slightly higher coefficients of permeability than those using river gravel for the w/c-ratios of 0.33 and 0.38. However, for the w/c-ratio of 0.45, the coefficients of permeability of concretes with Brooksville aggregate are slightly lower than those with river gravel.
4. Cement type has only a minor effect on the coefficient of permeability. For a w/c-ratio of 0.33, cement Type III produces a slightly lower coefficient of permeability than cement Types I and II for all cases, but in some cases the differences are not significant at an α level of 0.05. Concretes made with these three cement types exhibit approximately the same coefficient of permeability for the w/c-ratios of 0.38 and 0.45.
5. The use of smaller maximum aggregate size results in concretes with lower coefficient of permeability than those with larger maximum aggregate size. However, the coefficients of permeability of concretes using $3/8$ -inch and $3/4$ -inch maximum

aggregate sizes are not significantly different from one another at an α level of 0.05.

6. The coefficient of permeability of fly ash concrete is lower than the control concrete for all cases, except for the AC and CC curing conditions, at which the fly ash concrete with a water-cementitious ratio of 0.45 has a higher coefficient of permeability than the control concrete of the same condition. The differences in coefficients of permeability of these two concretes are not significant at an α level of 0.05 for some cases.
7. The addition of silica fume results in the reduction of coefficient of permeability by about 50% as compared with the control concrete. The silica fume concrete reaches its minimum coefficient of permeability at a silica fume addition of around 10% by weight of cement, but the differences are not significant at an α level of 0.05 for all percent additions of silica fume.
8. It can be noted that factors that contribute to an increase in the compressive strength also contribute to a reduction of the coefficient of permeability of concrete.
9. The degree of saturation of test specimens has some effect on the measured coefficient of permeability. The partially saturated specimens, which were used for this study, show a lower coefficient of permeability than the vacuum-saturated specimens in the range 0% to 30%. The average of the differences is about 12%.

CHAPTER 11 RECOMMENDED MIX DESIGN

11.1 Introduction

The results of an extensive laboratory study on the properties of Florida structural concretes have been presented in the previous four chapters. This chapter presents the development of a recommended mix design which minimizes permeability and maximizes strengths. First, the results from the laboratory testing program were used to select an initial optimum mix design. Then, adjustments of the mix proportion of the initial optimum mix design were tried in order to improve its strengths and impermeability. Finally, the mix design that resulted in the lowest permeability and highest strengths and was suitable for use in practice was determined. This final mix design is called the recommended "super mix."

11.2 Initial Optimum Mix

The test results from the previous chapters showed that the addition of silica fume improves the strengths and reduces the permeability of Florida Class IV concretes (concretes made with Brooksville aggregate). Therefore, the initial optimum mix design is to be selected from the silica fume concrete. Table 11.1 presents the mean compressive strengths, splitting tensile strengths, moduli of rupture and coefficients of permeability of the silica fume concrete. The silica fume concrete with 10% addition is selected as the initial optimum mix

Table 11.1 Results of Tests on Silica Fume Concrete

Percent Silica Fume	Curing* Condition	Mean Compressive Strength (psi)	Mean Splitting Tensile Strength (psi)	Mean Modulus of Rupture (psi)	Mean Coefficient of Permeability ($\times 10^{-12}$ in/sec)
0	28-MC	8459	629	735	3.85
5		9070	658	741	2.06
10		9372	593	775	1.59
15		9200	640	753	1.88
20		9142	629	802	1.60
0	90-MC	9081	695	779	3.09
5		9628	684	849	1.98
10		9823	736	877	1.65
15		9539	658	899	1.70
20		9452	653	960	1.45

Note: * Curing condition:

28-MC = 28-day moist curing

90-MC = 90-day moist curing

design. The reasons for the selection of 10% silica fume addition rather than 15% or 20% addition are as follows:

1. The 10% addition of silica fume exhibits the highest compressive strength at both 28-day and 90-day moist curing.
2. The 10% addition of silica fume produces the highest splitting tensile strength at 90-day moist curing. Although, at 28-day moist curing, the splitting tensile strength of silica fume concrete with 10% addition is slightly lower than that with 0%, 5%, 15% and 20% additions, the difference is not significant at an α level of 0.05.
3. The 10% addition of silica fume results in lower permeability than the other four percentages of addition at 28-day moist curing and results in only slightly higher permeability than that with 20% addition of silica fume at 90-day moist curing.
4. Although the 10% addition of silica fume produces lower modulus of rupture than the 20% addition, the moduli of rupture produced by these 2 levels of percent additions are not significantly different from each other at an α level of 0.05.
5. It is more preferable to use 10% silica fume addition than 20% silica fume addition in order to reduce the cost of the silica fume concrete.
6. The amount of sand in the concrete is reduced with the addition of silica fume. Therefore, the concrete with high percent addition of silica fume would lack a sufficient amount of sand which can cause some difficulties in the finishing of the concrete.

7. The concrete with higher percent addition of silica fume tends to lose workability faster than that with a lower percent addition of silica fume.

The silica fume concrete with 10% addition shows an improvement in compressive strength by about 11% and 8% from the control concrete at 28-day and 90-day moist curing, respectively. The modulus of rupture of the silica fume concrete is higher than the control concrete by about 5% and 12% at 28-day and 90-day moist curing, respectively. At 28-day moist curing, the splitting tensile strength of silica fume concrete is lower than the control concrete by about 6%, but at 90-day moist curing the silica fume concrete shows higher splitting tensile strength than the control concrete by about 6%. In terms of permeability, the silica fume concrete exhibits lower coefficient of permeability than the control concrete by about 59% and 47% at 28-day and 90-day moist curing, respectively. The use of silica fume concrete as structural concretes will have superior performance than that of the control concrete. This is because the silica fume concrete produces higher strengths which can reduce size of structural member in high-rise building or increase the span lengths for bridge girders. Also, the lower permeability of silica fume concrete can improve the durability of the concrete structure. Therefore, the silica fume concrete with 10% addition is selected as the initial optimum mix.

The mix combinations for this optimum mix design are as follows:

Cement Type II	=	752 lbs/yd ³
Silica Fume (10% by weight of cement)	=	75 lbs/yd ³
Water	=	273 lbs/yd ³
Brooksville #67 (dry weight)	=	1674 lbs/yd ³

Sand (dry weight)	= 1022 lbs/yd ³
Water-Cementitious Ratio	= 0.33.

The properties of coarse and fine aggregates can be found in Chapter 5.

11.3 Adjusted Optimum Mix

This section presents the mix proportions and results for the trial batch, which was made by adjusting the gradation of the blended aggregate (coarse and fine aggregates) of the initial optimum mix design. The blended aggregate of the initial optimum mix consisted of 62% (by weight) coarse aggregate (Brooksville #67) and 38% (by weight) fine aggregate. The gradation of the blended aggregate is shown in Figure 11.1.

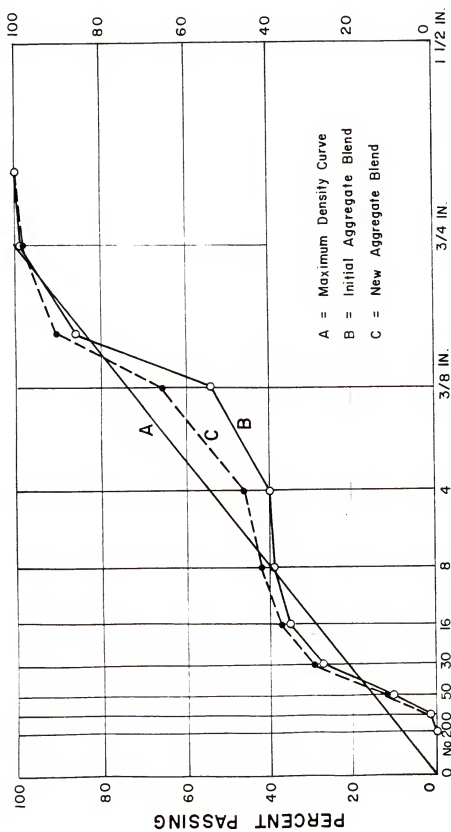
The adjustment of the gradation was done in the hope that a maximum density could be achieved. This means that the void volume in the concrete can be minimized, and the concrete strength of the trial batch can be increased. The maximum density curve from the maximum density equation using the 0.45 power was used as ideal grading curve. This ideal gradation is plotted along with the gradation of the aggregate blend of the initial optimum mix in Figure 11.1. The maximum density equation is expressed as follows [40]:

$$P_t = \left(\frac{d}{D}\right)^{0.45} \quad (11.1)$$

where P_t = total percentage passing given sieve

d = size of sieve opening

D = largest size in gradation.



SIEVE SIZES

Figure 11.1 Gradation Chart for Aggregate Blend (Coarse and Fine Aggregate)

It can be seen from Figure 11.1 that the gradation of the initial aggregate blend does not match very well with the maximum density curve, especially at the sieve sizes of $\frac{3}{8}$ inch and no. 4. This is because the distribution of the sizes of the coarse aggregate is concentrated in the range of $\frac{3}{8}$ inch and no. 4 sieve. In order to improve the gradation of the optimum mix, the Brooksville #89 was used as a partial replacement of Brooksville #67.

A blending of 45% of Brooksville #67, 15% of Brooksville #89 and 40% of sand was tried. The gradation of the new blended aggregate is also shown in Figure 11.1. It can be seen that the new gradation curve matches the maximum density curve better than the old one. A trial batch was made by using the new aggregate blend. The mix proportions for the trial batch are as follows:

Cement Type II	=	752 lbs/yd ³
Silica Fume (10% by weight of cement)	=	75 lbs/yd ³
Water	=	273 lbs/yd ³
Brooksville #67 (dry weight)	=	1213 lbs/yd ³
Brooksville #89 (dry weight)	=	405 lbs/yd ³
Sand (dry weight)	=	1078 lbs/yd ³
Water-Cementitious Ratio	=	0.33.

The test results of the trial batch at 28-day and 90-day moist curing are presented in Table 11.2. The comparison of means of strengths and permeability between the results from the initial and the adjusted optimum mixes are displayed in Table 11.3. It can be seen that the adjustment of the gradation of the aggregate blend improves the compressive strength and splitting tensile strength at 28-day moist curing

Table 11.2 Results of Tests on the Adjusted Optimum Mix

Curing* Condition	Speci- men #	Compressive Strength (psi)	Splitting Tensile Strength (psi)	Modulus of Rupture (psi)	Coefficient of Permeability ($\times 10^{-12}$ in/sec)
28-MC	1	9717	671	701	1.86
	2	9802	768	793	1.80
	Ave.	9760	720	747	1.84
90-MC	1	9745	680	766	2.29
	2	9851	735	761	-
	Ave.	9798	708	764	2.29

Note: * See footnotes on Table 11.1

Table 11.3 Comparison of Mean Compressive Strengths, Splitting Tensile Strengths, Moduli of Rupture and Coefficients of Permeability of the Initial Optimum Mix and Those of the Adjusted Optimum Mix

Properties of Concrete	28-Day Moist Curing			90-Day Moist Curing		
	A	B	Percent Difference	A	B	Percent Difference
Mean Compressive Strength (psi)	9372	9760	4	9823	9798	-0.2
Mean Splitting Tensile Strength (psi)	593	720	21	736	708	-4
Mean Moduli of Rupture (psi)	775	747	-4	877	764	-13
Mean Coefficient of Permeability ($\times 10^{-12}$ in/sec)	1.59	1.84	-15	1.65	2.29	-38

Note: A = Results from the initial optimum mix

B = Results from the adjusted optimum mix

by about 4% and 21%, respectively. At 90-day moist curing, the strengths of the adjusted concrete are slightly lower than the strengths of the initial mix. The permeability of the adjusted mix is slightly higher than the initial mix at both 28-day and 90-day moist curing.

From the test results presented above, the adjustment of the aggregate gradation of the initial optimum mix does not produce much improvement in strengths and reduction in permeability. Besides, the use of two coarse aggregates (aggregate #67 and #89) in the same mix would make the batching process more complicated. Therefore, the initial optimum mix design is chosen to be the recommended super mix in this study. The mix proportions for the recommended super mix can be found in Section 11.2.

The study of the mix design that gives higher strengths and lower permeability than the mix design of the super mix will be continued in Phase II of this study (to be conducted by Mr. Prasan Amornsrivilai), in which other adjustments to the optimum mix will be used in experiments to improve its strengths and impermeability.

CHAPTER 12
DEVELOPMENT OF PREDICTION EQUATIONS
FOR CONCRETE PROPERTIES

12.1 Introduction

This chapter presents the results of regression analyses to establish relationships between the various concrete properties obtained in this study. Regression analyses were performed to relate one concrete property to another. These best fit equations were also compared to the available equations from the ACI Code [36]. In addition, comments on the test results on Florida concrete are also presented.

12.2 Relationship Between Compressive
Strength and Modulus of Rupture

The compressive strength and modulus of rupture are closely related, but there is no direct proportionality [4]. In general, as the compressive strength increases, the modulus of rupture also increases. Many equations relating the compressive strength to the modulus of rupture have been developed. The ACI equation is one of the most commonly used ones in the field of structural concrete and is expressed as followed [36]:

$$f_r = 7.5 \sqrt{f'_c} \quad (12.1)$$

where f_r = modulus of rupture, psi

f'_c = compressive strength, psi

A regression equation of this form was used to perform the regression analysis in this study. Tabulated results in Tables C.1 and C.2 in Appendix C were used to plot and develop the relationship between the compressive strength and modulus of rupture. The relationship for concretes using Brooksville aggregate at 28-day moist curing, is shown in Figure 12.1.

Since aggregate type has some effect on the relationship between these two properties of concrete, it was considered appropriate to regress the data for each aggregate type separately and also for all three aggregate types combined. Data from the two curing conditions, namely 28-day and 90-day moist curing, were also treated separately. Regression analyses resulted in coefficients presented in Table 12.1. All the coefficients are higher than that of the ACI equation. Concretes using Calera aggregate produced higher regression coefficients than those using Brooksville or river gravel aggregate. This may be attributed to the good bonding characteristics in conjunction with the high strength of the Calera aggregate. Neville [5] stated that the relation between these two properties depends on the type of coarse aggregate used. This is because the properties of an aggregate affects the modulus of rupture more than the compressive strength. The results of this study are in agreement with Neville. For the concrete made with Brooksville aggregate, the regression coefficient is higher by about 8% as compared with that of the ACI equation. According to the results from this study, the use of ACI equation to estimate the modulus of rupture of Florida concrete (concrete made with Brooksville aggregate) will generate conservative and somewhat reasonable results. The curves

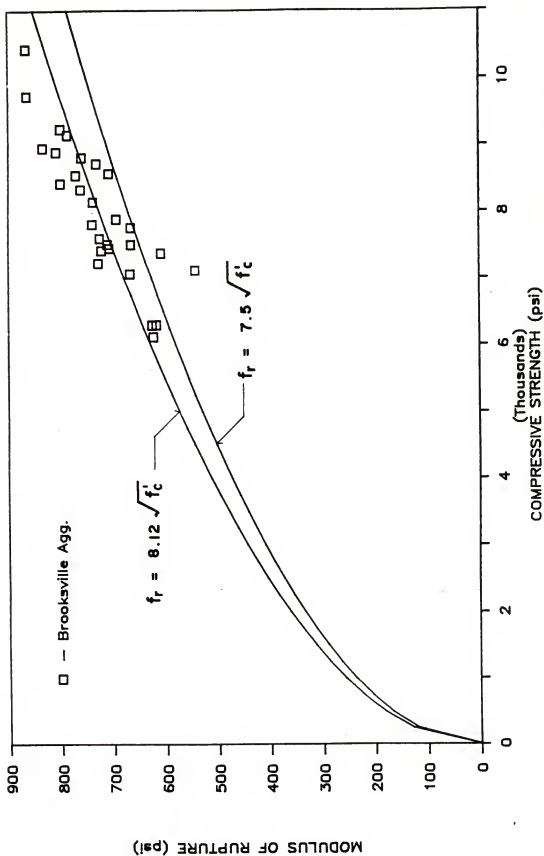


Figure 12.1 Relationship Between Compressive Strength and Modulus of Rupture of Concrete Made with Brooksville Aggregate at 28-Day Moist Curing

Table 12.1 Regression Coefficients for Equations Relating Compressive Strength to Modulus of Rupture

Equation	Curing Condition	Aggregate Type	Coefficient A	Standard Error
$f_r = A \sqrt{f'_c}$	28-day moist curing	Brooksville	8.12	0.095
		Calera	9.71	0.265
		River Gravel	9.06	0.246
		Combined	8.68	0.134
	90-day moist curing	Brooksville	8.20	0.092
		Calera	10.29	0.264
		River Gravel	9.59	0.301
		Combined	8.98	0.164

representing the prediction equation for Brooksville aggregate at 28-day moist curing and the ACI equation are also shown in Figure 12.1.

12.3 Relationship Between Compressive Strength and Splitting Tensile Strength

The compressive strengths as tabulated in Appendix C were plotted against the splitting tensile strength at the 28-day moist curing. The relation between these two properties for concrete using Brooksville aggregate is illustrated in Figure 12.2. Regression analyses to establish empirical relationship between compressive and splitting tensile strengths were performed using the following models:

$$f_{ct} = A \sqrt{f'_c} \quad (12.2)$$

$$f_{ct} = f'_c{}^B \quad (12.3)$$

where f_{ct} = splitting tensile strength (psi)

A and B = coefficients

The ACI Code uses Equation 12.2 for estimation of splitting tensile strength of lightweight concrete, where the coefficient A is equal to 6.7 [36].

The investigation by Carino and Lew [58] determined that the coefficient A was approximately 6.49. They suggested that the estimation of the splitting tensile strength by using Equation 12.3 was better than Equation 12.2. For low compressive strengths, they found that Equation 12.2 resulted in an overestimation of the splitting tensile strength, while for the high compressive strengths this equation underestimated

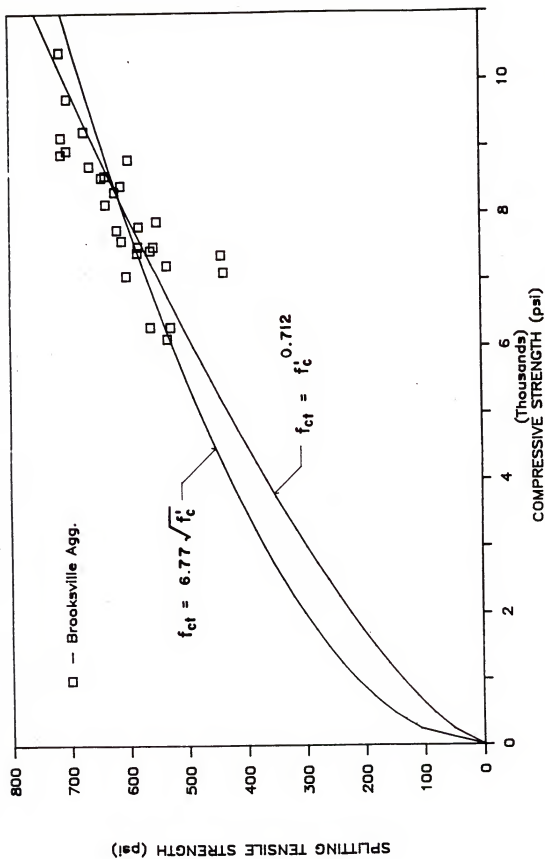


Figure 12.2 Relationship Between Compressive Strength and Splitting Tensile Strength of Concrete Made with Brooksville Aggregate at 28-Day Moist Curing

the splitting tensile strength. The coefficient B in Equation 12.3 was determined to be approximately 0.73 in their investigation.

Regression analyses were performed on the experimental data for each aggregate type and for each of the two curing conditions (28-day and 90-day moist curing). The results are summarized in Table 12.2. It can be seen that concretes using Calera aggregate produced higher coefficients (A and B) than those using Brooksville or river gravel aggregate for both equations. These results show a trend similar to the previous case (Section 12.2) for estimation of modulus of rupture. The coefficient A for all three aggregate types are higher than both the ACI and Carino and Lew coefficients. The ACI coefficient is about the same as that of the concrete using Brooksville aggregate.

The coefficient B from this study is about the same as that from the Carino and Lew investigation. Figure 12.2 also shows the curves representing Equations 12.2 and 12.3 for concrete using Brooksville aggregate at 28-day moist curing. It can be seen that Equation 12.3 shows a better fit to the data than Equation 12.2, especially for compressive strengths higher than 8000 psi. Equation 12.3 is suggested for use, if the estimation of the splitting tensile strength is needed. Equation 12.3 will result in lower values than Equation 12.2, when the compressive strength is lower than 8000 psi.

12.4 Relationship Between Compressive Strength and Coefficient of Permeability

The compressive strengths and coefficients of permeability at 28-day moist curing as displayed in Tables C.2 and C.4 were plotted in Figure 12.3 to observe the relationship between the compressive strength and coefficient of permeability. It can be seen that the higher the

Table 12.2 Regression Coefficients for Equations Relating Compressive Strength to Splitting Tensile Strength

Equation	Curing Condition	Aggregate Type	Coefficient A or B	Standard Error
$f_{ct} = A \sqrt{f'_c}$	28-day moist curing	Brooksville	6.77	0.104
		Calera	7.62	0.177
		River Gravel	6.97	0.104
		Combined	7.01	0.088
	90-day moist curing	Brooksville	7.12	0.083
		Calera	7.95	0.098
		River Gravel	7.24	0.108
		Combined	7.34	0.075
$f_{ct} = f'_c{}^B$	28-day moist curing	Brooksville	0.712	0.0018
		Calera	0.721	0.0022
		River Gravel	0.714	0.0019
		Combined	0.714	0.0013
	90-day moist curing	Brooksville	0.717	0.0014
		Calera	0.724	0.0012
		River Gravel	0.717	0.0017
		Combined	0.718	0.0010

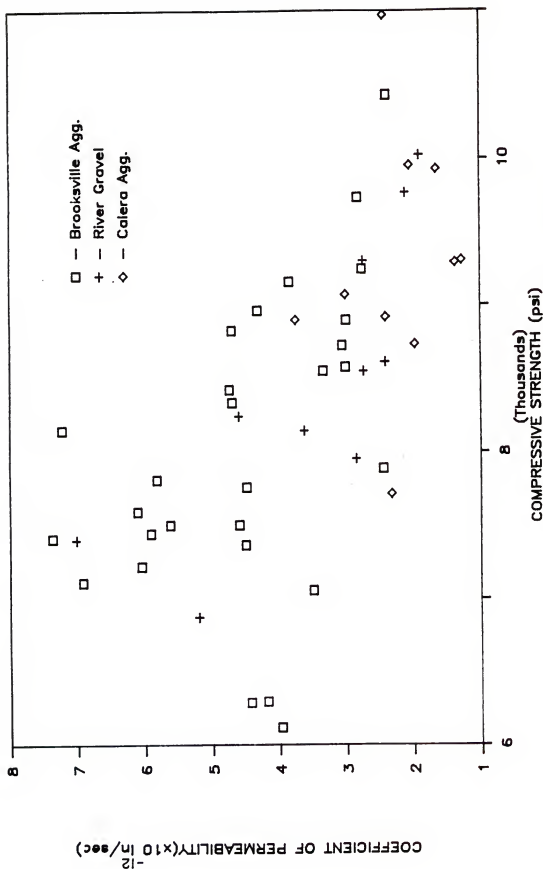


Figure 12.3 Relationship Between Compressive Strength and Coefficient of Permeability at 28-Day Moist Curing

compressive strength, the lower the permeability would be, but there is not a clear relationship between these two properties. The following three forms of regression equations were used to investigate the relationship between these two properties:

$$K = A + B f'_C + C (f'_C)^2 \quad (12.4)$$

$$K = A + B \log f'_C + C (\log f'_C)^2 \quad (12.5)$$

$$\log k = A + B f'_C \quad (12.6)$$

where K = coefficient of permeability ($\times 10^{-12}$ in/sec)

k = coefficient of permeability (in/sec)

A, B and C = coefficients

Regression analyses were performed between the compressive strengths and the coefficients of permeability. The results are summarized in Table 12.3. It can be seen that the coefficient of permeability correlated poorly with the compressive strength for concretes using Calera and Brooksville aggregates and correlated moderately well for concretes using river gravel. Equation 12.6 gives slightly better correlation than Equations 12.4 and 12.5 for most of the cases. R -squared values range from 0.078 (for Calera aggregate) to 0.73 (for river gravel) at 28-day moist curing.

From the results of these analyses, the relationship between the compressive strength and coefficient of permeability is dependent on the type of aggregate used. The type of aggregate should be considered in the selection of the prediction equation for the coefficient of permeability from the compressive strength. For design purpose, it is not

Table 12.3 Regression Coefficients for Equations Relating Compressive Strength to Coefficient of Permeability

Equation	Curing Condition	Aggregate Type	Regression Coefficients			R ²
			A	B	C	
$K = A + B f'_c + C (f'_c)^2$	28-Day Moist Curing	Brooksville	-14.19	0.0053	-3.69×10^{-7}	0.341
		Calera	11.69	-0.0019	9.27×10^{-8}	0.047
		River Gravel Combined	34.24	-0.006	2.74×10^{-7}	0.661
			3.84	0.00095	-1.12×10^{-7}	0.410
	90-Day Moist Curing	Brooksville	-26.00	0.0078	-5.01×10^{-7}	0.362
		Calera	-10.48	0.0029	-1.68×10^{-7}	0.455
		River Gravel Combined	56.30	-0.011	5.14×10^{-7}	0.820
			4.30	0.0007	-9.13×10^{-8}	0.503
$K = A + B \log f'_c + C (\log f'_c)^2$	28-Day Moist Curing	Brooksville	-2227.55	1156.74	-149.82	0.370
		Calera	405.76	-200.04	24.78	0.041
		River Gravel Combined	1119.75	-543.32	65.96	0.652
			-1160.09	612.63	-80.52	0.431
	90-Day Moist Curing	Brooksville	-3202.9	1647.91	-211.66	0.361
		Calera	-1153.63	587.34	-74.61	0.438
		River Gravel Combined	3474.84	-1729.23	215.24	0.811
			-945.52	499.45	-65.62	0.50
$\log k = A + B f'_c$	28-Day Moist Curing	Brooksville	-13.21	0.00053	-	0.398
		Calera	-8.714	-0.00061	-	0.078
		River Gravel Combined	-9.10	-0.00041	-	0.7338
			-11.40	0.0001	-	0.445
	90-Day Moist Curing	Brooksville	-16.09	0.0012	-	0.397
		Calera	-14.16	0.00059	-	0.445
		River Gravel Combined	-6.89	-0.00086	-	0.808
			-12.50	0.00035	-	0.572

recommended to any of these equations for estimation of permeability. Direct measurement of concrete permeability is recommended.

12.5 Relationship Between Modulus of Rupture and Coefficient of Permeability

The moduli of rupture at 28-day moist curing as displayed in Table C.1 were plotted against the coefficients of permeability at the same curing condition (shown in Table C.4). The relationship is shown in Figure 12.4. It can be seen that the relationship for these two properties is similar to the previous case (Section 12.4). There is no clear relationship. Regression analyses were performed using the regression equations similar to Equations 12.4 through 12.6. The results are summarized in Table 12.4. It can be seen that the relationship between these two properties is moderately well defined for concretes using river gravel and very poorly defined for concretes using Brooksville and Calera aggregates. These results are similar to the results for the previous case of estimation of permeability from compressive strength. Equation 12.6 also gives slightly better correlation than Equations 12.4 and 12.5 for most of the cases.

12.6 Relationship Between Splitting Tensile Strength and Coefficient of Permeability

The splitting tensile strengths and coefficients of permeability at 28-day moist curing from Tables C.3 and C.4 were plotted in Figure 12.5 to observe the relationship between these two properties. It can be observed that there is not a clear relationship. Generally, the higher the splitting tensile strength, the lower the permeability would be. This is also similar to the case of compressive strength. For the

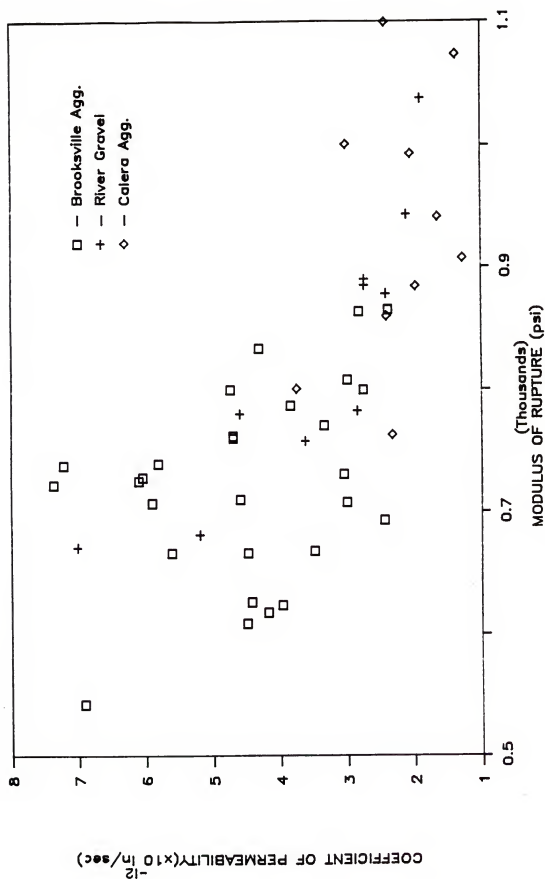


Figure 12.4 Relationship Between Modulus of Rupture and Coefficient of Permeability at 28-Day Moist Curing

Table 12.4 Regression Coefficient for Equations Relating Modulus of Rupture to Coefficient of Permeability

Equation	Curing Condition	Aggregate Type	Regression Coefficients			R ²
			A	B	C	
$K = A + B f_r + C (f_r)^2$	28-Day Moist Curing	Brooksville	-5.26	0.034	-2.87×10^{-5}	0.157
		Calera	20.71	-0.038	1.91×10^{-5}	0.187
		River Gravel Combined	41.614 13.82	-0.08 -0.016	4.03×10^{-5} 4.67×10^{-6}	0.863 0.460
	90-Day Moist Curing	Brooksville	-5.90	0.037	-3.15×10^{-5}	0.198
		Calera	22.84	-0.037	1.58×10^{-5}	0.454
		River Gravel Combined	31.29 18.33	-0.054 -0.028	2.43×10^{-5} 1.13×10^{-5}	0.645 0.459
$K = A + B \log f_r + C (\log f_r)^2$	28-Day Moist Curing	Brooksville	-393.90	290.49	-52.85	0.140
		Calera	719.17	-479.24	80.06	0.202
		River Gravel Combined	1161.03 -48.56	-770.93 52.08	128.20 -11.74	0.846 0.453
	90-Day Moist Curing	Brooksville	-988.03	708.79	-126.52	0.202
		Calera	958.18	-624.48	101.89	0.501
		River Gravel Combined	943.04 295.55	-617.65 -184.09	101.33 28.75	0.689 0.462
$\log k = A + B f_r$	28-Day Moist Curing	Brooksville	-12.55	0.0041	-	0.189
		Calera	-7.91	-0.0078	-	0.186
		River Gravel Combined	-8.26 -10.56	-0.0063 -0.0012	- -	0.891 0.530
	90-Day Moist Curing	Brooksville	-15.37	0.0119	-	0.239
		Calera	-7.179	-0.008	-	0.444
		River Gravel Combined	-9.208 -9.95	-0.004 -0.0026	- -	0.600 0.500

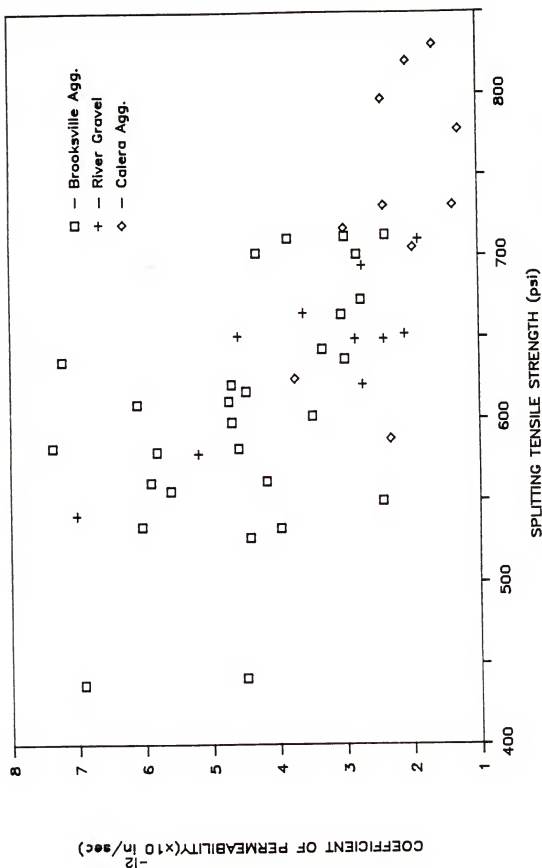


Figure 12.5 Relationship Between Splitting Tensile Strength and Coefficient of Permeability at 28-Day Moist Curing

investigation of the relationship between these two properties, regression analyses were performed using equations similar to Equations 12.4 through 12.6. Results of the analyses as summarized in Table 12.5, indicate that the coefficient of permeability correlated poorly with the splitting tensile strength for concretes using Brooksville and Calera aggregates. For concretes using river gravel, the relationship between these two properties is moderately well defined. These results also follow the same trend as observed in the case of compressive strength and modulus of rupture. Equation 12.5 shows slightly better correlation than Equations 12.4 and 12.6 for most of the cases.

12.7 Relationship Between Modulus of Rupture and Splitting Tensile Strength

According to Narrow and Ullberg [39], the modulus of rupture correlates very well with the splitting tensile strength. The higher the modulus of rupture, the higher the splitting tensile strength would be. The moduli of rupture and splitting tensile strengths at 28-day moist curing from Tables C.1 and C.3 in Appendix C were plotted in Figure 12.6 to observe the trend of the relationship.

Regression analyses were performed on the experimental data using the following linear regression model:

$$f_{ct} = A f_r \quad (12.7)$$

The results are summarized in Table 12.6. It can be seen that coefficient A for concretes using Brooksville aggregate is slightly higher than those using Calera or river gravel aggregate for both curing conditions. For concretes using the same aggregate type, the coefficients A

Table 12.5 Regression Coefficients for Equations Relating Splitting Tensile Strength to Coefficient of Permeability

Equation	Curing Condition	Aggregate Type	Regression Coefficients			R ²
			A	B	C	
$K = A + B f_{ct} + C (f_{ct})^2$	28-Day Moist Curing	Brooksville	-2.17	0.033	-3.62×10^{-5}	0.269
		Calera	6.29	-0.006	7.69×10^{-7}	0.283
		River Gravel Combined	82.14 9.64	-0.225 -0.005	1.59×10^{-4} -5.81×10^{-6}	0.757 0.461
	90-Day Moist Curing	Brooksville	77.55	-0.212	1.51×10^{-4}	0.346
		Calera	-4.16	0.0213	-1.73×10^{-5}	0.438
		River Gravel Combined	94.12 31.33	-0.244 -0.067	1.62×10^{-4} 3.73×10^{-5}	0.692 0.511
$K = A + B \log f_{ct} + C (\log f_{ct})^2$	28-Day Moist Curing	Brooksville	-498.57	377.49	-70.67	0.262
		Calera	-188.04	141.80	-26.31	0.261
		River Gravel Combined	2399.84 -260.81	-1679.81 208.20	294.25 -40.59	0.762 0.461
	90-Day Moist Curing	Brooksville	2860.51	-2008.92	353.09	0.346
		Calera	-527.68	377.43	-67.18	0.475
		River Gravel Combined	3922.80 769.50	-2729.49 -516.31	475.02 86.76	0.724 0.512
$\log k = A + B f_{ct}$	28-Day Moist Curing	Brooksville	-12.20	0.0039	-	0.301
		Calera	-10.55	-0.0022	-	0.262
		River Gravel Combined	-5.68 -11.33	-0.016 0.0012	-	0.656 0.522
	90-Day Moist Curing	Brooksville	-3.54	-0.023	-	0.276
		Calera	-12.62	0.0037	-	0.458
		River Gravel Combined	-4.15 -9.23	-0.019 -0.005	-	0.588 0.509

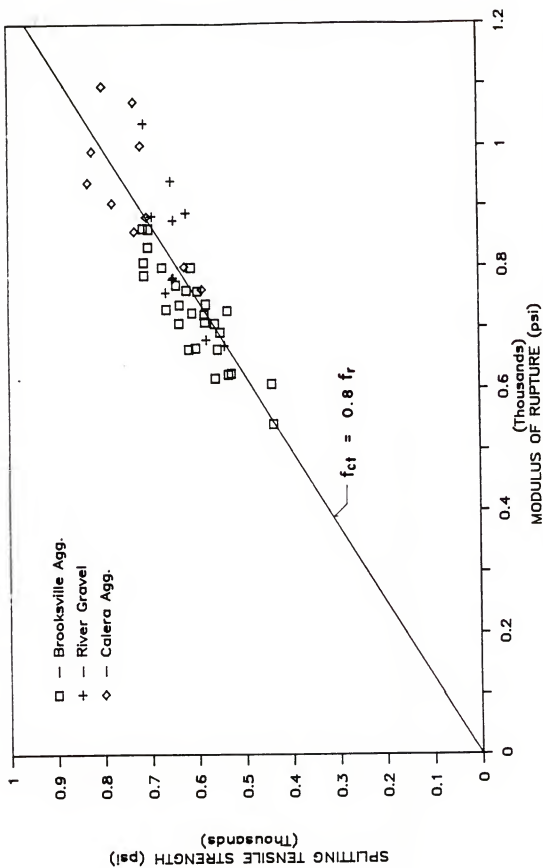


Figure 12.6 Relationship Between Modulus of Rupture and Splitting Tensile Strength at 28-Day Moist Curing

Table 12.6 Regression Coefficients for Equations Relating Modulus of Rupture to Splitting Tensile Strength

Equation	Curing Condition	Aggregate Type	Coefficient A	Standard Error
$f_{ct} = A f_r$	28-Day Moist Curing	Brooksville	0.83	0.010
		Calera	0.78	0.022
		River Gravel	0.76	0.023
		Combined	0.80	0.010
	90-Day Moist Curing	Brooksville	0.86	0.011
		Calera	0.77	0.022
		River Gravel	0.75	0.019
		Combined	0.81	0.012

for the 28-day and 90-day moist-curing conditions are approximately the same. Narrow and Ullberg [39] found that the relationship between these two properties is not affected by age of the specimen when tested and the aggregate type used. The results from this study tend to support the results of their study.

Equation 12.1 which relates compressive strength to modulus of rupture, can be combined with Equation 12.2, which relates compressive strength to splitting tensile strength, to produce Equation 12.7. If the ACI coefficient of 6.7 and coefficient of 6.49 from Carino and Lew investigation (see Section 12.3) are used for the coefficient in Equation 12.2, the derived coefficients A in Equation 12.7 will be equal 0.893 and 0.865, respectively. These two coefficients are higher than the results from this study by about 10%.

Troxell et al. [10] stated that the splitting tensile strength ranges approximately from 50% to 75% of modulus of rupture. In this study, the splitting tensile strength is about 75% to 85% of modulus of rupture, and is higher than the range given by Troxell et al. According to the literature review by Nielsen [59], the length of the beam specimen affects the modulus of rupture. For specimens with constant cross-sectional area, the specimen with longer length exhibited lower modulus of rupture than that with shorter length. Malhotra [60] found that the splitting tensile strength of smaller sized specimen is higher than that with larger sized specimen. The use of different specimen sizes and lengths should result in different ratios. This factor might be one of the reasons contributing to the difference in results. The plot of the regression equation for combination of three aggregate types at 28-day moist curing is also shown in Figure 12.6. Since the relationship

between modulus of rupture and splitting tensile strength is a linear relationship, it is appropriate to use the splitting tensile strength to estimate the modulus of rupture.

12.8 Relationship Between Compressive Strength and Modulus of Elasticity

The moduli of elasticity were measured from the cylindrical specimens which had been moist cured for 28 and 90 days. A compressometer was used to measure the deformation of the cylindrical specimen as it was loaded in the compressive strength test, as described in Chapter 4.

According to the ACI Code [36], the approximate relationship between the compressive strength and modulus of elasticity is as follows:

$$E = 33 w^{1.5} \sqrt{f'_c} \quad (12.8)$$

where E = modulus of elasticity, psi

w = unit weight of concrete, lbs/ft³

This form of equation was used as a model in this study. Regression analyses were performed on the experimental data as shown in Tables C.2 and C.5 in Appendix C. The results are summarized in Table 12.7. It can be seen that the coefficient A for concretes using Calera aggregate is higher than those using Brooksville or river gravel aggregate for both curing conditions. The coefficient A for the case of Brooksville and river gravel aggregates are about the same.

Coefficients A for all three aggregate types are higher than the ACI equation by about 3% to 30%. Therefore, the use of the ACI equation

Table 12.7 Regression Coefficients for Equations Relating Compressive Strength to Modulus of Elasticity

Equation	Curing Condition	Aggregate Type	Coefficient A	Standard Error
$E = A w^{1.5} \sqrt{f'_c}$	28-Day Moist Curing	Brooksville	34.05	0.198
		Calera	40.31	0.444
		River Gravel	34.23	0.178
		Combined	35.65	0.314
	90-Day Moist Curing	Brooksville	34.16	0.203
		Calera	40.09	0.472
		River Gravel	33.35	0.309
		Combined	35.51	0.325

to estimate the modulus of elasticity of Florida concrete will result in conservative estimates.

12.9 Relationship Between Modulus of Rupture and Modulus of Elasticity

The moduli of elasticity examined in this section were determined by means of strain gages mounted on the beam specimens, which had been moist cured for 28 and 90 days. The results are presented in Table C.5 in Appendix C.

There is no direct equation for the relationship between modulus of rupture and modulus of elasticity in the ACI Code [36]. However, the approximate relationship between these two properties can be obtained by combining Equations 12.2 and 12.8. The combined equation is as follows:

$$E = 4.4 w^{1.5} f_r \quad (12.9)$$

This form of equation was used as a model to develop the relationship in this study. Regression analyses were performed on the modulus of rupture and modulus of elasticity data. The results are summarized in Table 12.8. It can be seen that coefficient A for the case of the Brooksville aggregate is higher than those for the other two aggregate types. However, at 28-day moist curing, concretes with the same modulus of rupture, the modulus of elasticity of concrete with Calera aggregate is slightly higher than that with Brooksville aggregate. This is because the unit weight of concretes with Calera aggregate is higher than that with Brooksville aggregate.

Coefficients A for all three aggregate types are lower than the coefficient in Equation 12.9. The difference in the coefficient may be

Table 12.8 Regression Coefficients for Equations Relating Modulus of Rupture to Modulus of Elasticity

Equation	Curing Condition	Aggregate Type	Coefficient A	Standard Error
$E = A w^{1.5} f_r$	28-Day Moist Curing	Brooksville	4.20	0.051
		Calera	4.09	0.106
		River Gravel	3.69	0.107
		Combined	4.04	0.049
	90-Day Moist Curing	Brooksville	4.19	0.052
		Calera	3.82	0.087
		River Gravel	3.64	0.060
		Combined	3.94	0.043

due to the fact that the modulus of elasticity in Equation 12.8 was determined from the compression test rather than the beam test.

12.10 Relationship Between Compressive Strength and Schmidt Hammer Rebound Number

Schmidt hammer test is one of the most commonly used nondestructive tests. The test measures the rebound of a harder steel hammer impacted on the concrete by a spring [40]. In reality, this test indicates the hardness of the concrete surface [5]. There is not any direct relationship between the compressive strength and the rebound number. In this study, the Schmidt hammer test was run horizontally on all specimens before the strength test was performed. The experimental data were used to develop the empirical relationship.

Compressive strengths as shown in Table C.1 and rebound numbers as shown in Table C.8 in Appendix C were used to develop the relationship between compressive strength and rebound number. After preliminary analyses were performed, two regression models were found to fit the data better than the other models. These two models are a linear equation and a second degree polynomial (quadratic) equation. Regression analyses were performed between compressive strength and rebound number. Regression coefficients for this relationship are presented in Table 12.9. It can be seen that R-squared values for both models are about the same for all cases, and the range is from 0.74 to 0.87.

Linear regression lines for the three different aggregate types are presented in Figure 12.7. It can be seen that the aggregate type affects the rebound number. For the same rebound number, concretes using Calera aggregate exhibited higher strength than those using Brooksville or river gravel aggregate. For the rebound numbers from 30

Table 12.9 Regression Coefficients for Equations Relating Schmidt Hammer Rebound Number to Compressive Strength

Equation	Curing Condition	Aggregate Type	Regression Coefficients			R ²
			A	B	C	
$f'_c = A + Bx$ (Linear Equation)	28-Day Moist Curing	Brooksville	-1456.07	245.44	-	0.78
		Calera	-3189.25	308.70	-	0.74
		River Gravel	-7080.27	394.99	-	0.87
	Combined All 6 Curing Conditions	Brooksville Calera River Gravel Combined	-3672.92 -5293.94 -5995.57 -4680.56	304.11 359.52 364.05 334.23	- - - -	0.87 0.81 0.83 0.85
$f'_c = A + Bx + Cx^2$ (Polynomial Equation)	28-Day Moist Curing	Brooksville	3486.19	-23.39	3.63	0.78
		Calera	102519.43	-4868.08	63.24	0.83
		River Gravel	8327.77	-394.42	10.08	0.87
	Combined All 6 Curing Conditions	Brooksville Calera River Gravel Combined	-8036.92 10615.97 -16183.70 -2792.33	554.19 -472.5 906.15 228.75	-3.53 10.79 -7.15 1.45	0.87 0.83 0.80 0.85

Note: x = Rebound number

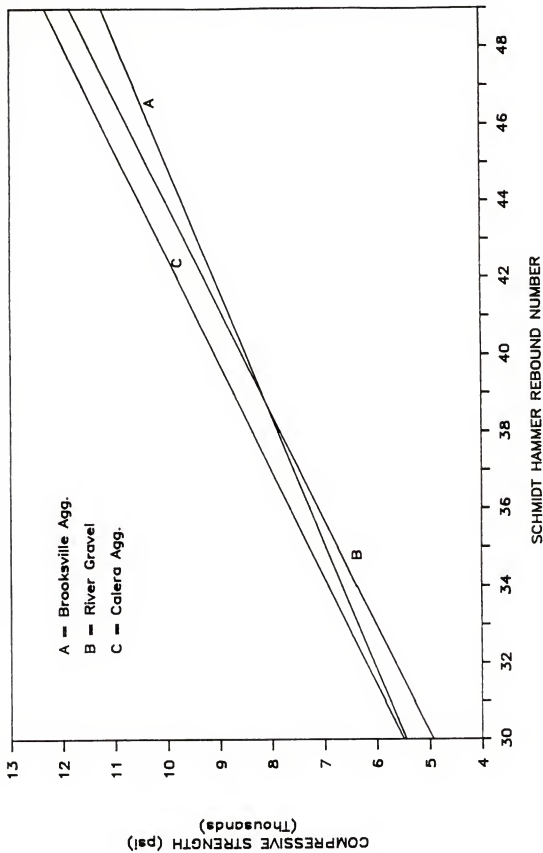


Figure 12.7 Relationship Between Compressive Strength and Schmidt Hammer Rebound Number

to 39, concretes using Brooksville aggregate showed higher strength than those using river gravel, while the results are reversed when the rebound numbers are higher than 39.

12.11 Relationship Between Modulus of Rupture and Schmidt Hammer Rebound Number

The Schmidt hammer test was also run horizontally on all beam specimens before the modulus of rupture tests were performed. The experimental data for modulus of rupture and the Schmidt hammer test are presented in Tables C.1 and C.7, respectively. Preliminary analyses revealed that the relationship between modulus of rupture and rebound number was better defined by the linear regression and quadratic regression models than the other models.

Regression analyses were performed on the experimental data using the two models mentioned above. The results are summarized in Table 12.10. It can be seen that R-squared values from the polynomial regression model are slightly higher than those from the linear regression model. R-squared values range from 0.48 to 0.77. The relationship between modulus of rupture and rebound number can be defined by a second degree polynomial expression with the independent variable being the rebound number.

Figure 12.8 shows the plots of the three regression equations for the three aggregate types. It can be seen that the rebound numbers are affected by the type of aggregate used. For the same rebound number, the Calera aggregate gave higher modulus of rupture than the Brooksville or the river gravel aggregate. The modulus of rupture of concrete using Brooksville aggregate are about the same as those using river gravel, for rebound numbers ranging from 35 to 38. However, the river gravel

Table 12.10 Regression Coefficients for Equations Relating Schmidt Hammer Rebound Number to Modulus of Rupture

Equation	Curing Condition	Aggregate Type	Regression Coefficients			R ²
			A	B	C	
$f_r = A + Bx$ (Linear Equation)	28-Day Moist Curing	Brooksville	-31.11	18.40	-	0.71
		Calera	-828.54	41.22	-	0.64
		River Gravel	-901.96	40.13	-	0.77
	Combined All 6 Curing Conditions	Brooksville Calera River Gravel Combined	124.72 -672.65 -503.58 -208.62	14.12 36.89 30.38 23.56	- - - -	0.43 0.60 0.69 0.52
$f_r = A + Bx + Cx^2$ (Polynomial Equation)	28-Day Moist Curing	Brooksville	1154.82	-42.19	0.77	0.73
		Calera	2172.51	-98.84	1.63	0.64
		River Gravel	-25.30	0.258	0.45	0.77
	Combined All 6 Curing Conditions	Brooksville Calera River Gravel Combined	1388.45 5181.54 2742.49 2263.05	-54.15 -244.14 -129.25 -105.56	0.91 3.35 1.95 1.66	0.48 0.67 0.76 0.60

Note: x = Rebound number

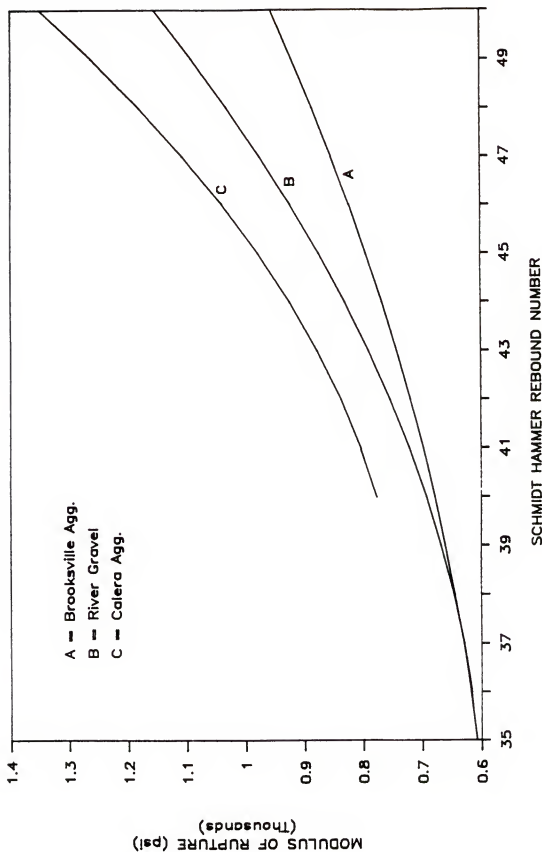


Figure 12.8 Relationship Between Modulus of Rupture and Schmidt Hammer Rebound Number

gave higher modulus of rupture than the Brooksville aggregate for the rebound numbers higher than 38. These results show trends similar to that for the case of compressive strength.

12.12 Relationship Between Splitting Tensile Strength and Schmidt Hammer Rebound Number

The Schmidt hammer test was run on all cylindrical specimens before the splitting tensile test was performed. The direction of hammer impact was horizontal. The splitting tensile strengths as shown in Table C.3 and rebound numbers as shown in Table C.9 in Appendix C were used to develop a relationship between splitting tensile strength and rebound number. Preliminary analyses revealed that relationships defined by the linear regression and second degree polynomial regression models would fit the data better than the other models.

The experimental data were regressed using the two models mentioned above. The analyses resulted in coefficients which are presented in Table 12.11. It can be seen that the R-squared values from the polynomial regression model are slightly higher than those from the linear regression model. R-squared values range from 0.39 to 0.80. The relatively low R-squared values for this case as compared with those for compressive strength and modulus of rupture may have been caused by the limited area for performing the Schmidt hammer test. In order to keep the same direction of impact, only two lines along the sides of cylindrical specimens could be used, while for the beam specimens and cylindrical compression, the test could be performed at all areas around the specimens. Therefore, the rebound numbers from beam specimens and compression test specimens which represented the hardness of the total area showed less discrepancy as compared with the rebound numbers from the

Table 12.11 Regression Coefficients for Equations Relating Schmidt Hammer Rebound Number to Splitting Tensile Strength

Equation	Curing Condition	Aggregate Type	Regression Coefficients			R ²
			A	B	C	
$f_{ct} = A + Bx$ (Linear Equation)	28-Day Moist Curing	Brooksville	83.53	14.78	-	0.33
		Calera	-375.17	30.87	-	0.62
		River Gravel	133.61	14.30	-	0.43
$f_{ct} = A + Bx + Cx^2$ (Polynomial Equation)	28-Day Moist Curing	Combined	-25.88	18.12	-	0.60
		All 6	-410.51	31.623	-	0.79
		River Gravel	-151.83	21.91	-	0.68
		Combined	-155.50	22.61	-	0.64
$f_{ct} = A + Bx + Cx^2$ (Polynomial Equation)	28-Day Moist Curing	Brooksville	2533.59	-133.19	2.22	0.39
		Calera	-6894.74	385.64	-4.82	0.64
		River Gravel	-1285.69	94.15	-1.12	0.44
$f_{ct} = A + Bx + Cx^2$ (Polynomial Equation)	28-Day Moist Curing	Brooksville	204.74	3.40	0.23	0.60
		Calera	735.25	-34.33	0.94	0.80
		River Gravel	-262.53	28.37	-0.09	0.68
$f_{ct} = A + Bx + Cx^2$ (Polynomial Equation)	28-Day Moist Curing	Combined	352.44	-9.03	0.49	0.65

Note: x = Rebound number

splitting tensile test specimens which represented the hardness of a limited area.

Figure 12.9 presents the second degree polynomial regression lines for each aggregate type. Data for all six curing conditions were used. It can be seen that the type of aggregate used influences the rebound number. The Calera aggregate gave a higher splitting tensile strength than the Brooksville or river gravel aggregate for the same rebound number. For rebound numbers ranging from 25 to 31, concretes using Brooksville aggregate exhibited higher tensile strength than those using river gravel, while the reverse is true for rebound numbers higher than 31. These results also show trends similar to those for the case of the compression test. The appropriate equation (according to the aggregate type) should be used for estimating the splitting tensile strength from the rebound number.

12.13 Comments on Test Results

Results of regression analyses to establish relationships between different concrete properties have been presented in the previous sections. In this section, comments on test results for each property of Florida concrete is presented. The test results are compared with the standard codes.

12.13.1 Compressive Strength

According to the FDOT Specifications [52], the required minimum 28-day compressive strengths for Florida Classes II, III and IV concretes are 3400, 5000 and 5500 psi, respectively. The test results in this study, which are presented in Table C.2 in Appendix C, exceed this requirements for all batches. For 28-day moist curing, the compressive

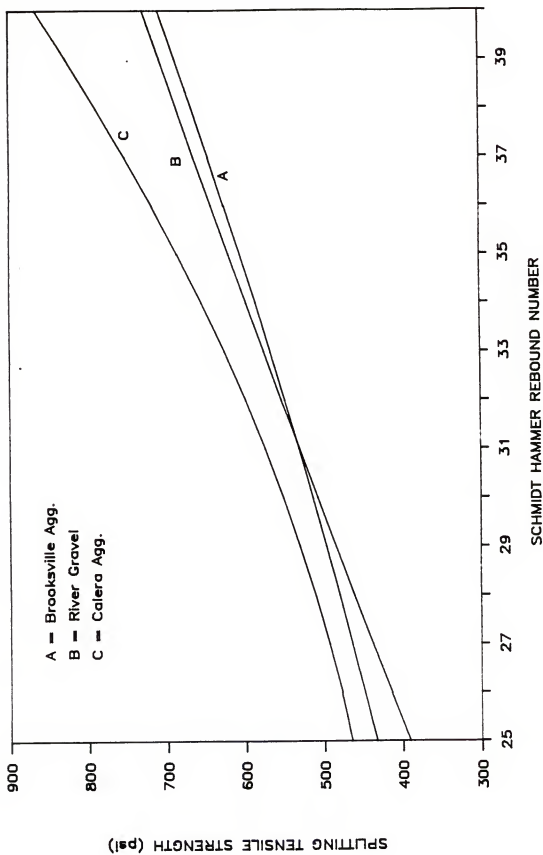


Figure 12.9 Relationship Between Splitting Tensile Strength and Schmidt Hammer Rebound Number

strengths of concretes using Brooksville aggregate are in the range 6000-9000 psi, which is considered to be in the range of strength for a high-strength concrete [30]. The compressive strength of silica fume concrete made with Brooksville aggregate is in the range 9000-9600 psi at 28-day moist curing.

From the test results in this study, it can be concluded that the Brooksville aggregate can produce a high-strength concrete with the compressive strength in the range 6000-9000 psi. The use of high-range water reducer may be needed in order to achieve this goal. For strength higher than 9000 psi, the addition of silica fume and the use of high-range water reducer are recommended.

12.13.2 Modulus of Rupture

There is no minimum requirement on modulus of rupture of Florida concrete. According to Texas specifications, the minimum acceptable 7-day center-point loading modulus of rupture is 600 psi [48]. Wright and Harwood [35] found that the modulus of rupture under central loading was 20% to 25% higher than that under third-point loading. Thus, the modulus of rupture of 600 psi under central loading is equal to about 500 psi when tested under third-point loading. The test results from this study show that the moduli of rupture of 7-day moist-cured specimens are higher than 500 psi for all batches. Therefore, all Florida concretes in this study meet the minimum 7-day modulus of rupture requirements of Texas specifications.

In general, the compressive strength is used to estimate the modulus of rupture. The empirical relationship between compressive strength and modulus of rupture suggested by the ACI Code is the most commonly used. This relationship is expressed in Equation 12.1. In this study,

the modulus of rupture of concrete using Brooksville aggregate is higher than the modulus of rupture as estimated by using Equation 12.1 by about 8% for 28-day moist curing condition.

Since the modulus of rupture of concrete using Brooksville aggregate is higher than the minimum requirement of Texas specifications and the ACI Code, the use of Brooksville aggregate as a coarse aggregate for Florida concrete is suitable for concrete structures where control of modulus of rupture is important.

12.13.3 Splitting Tensile Strength

The test results from this study show that the splitting tensile strengths of concretes at 28-day moist curing vary in a range of 425-760 psi for Brooksville aggregate, while for Calera and river gravel aggregates, the tensile strengths are in the ranges of 580-885 and 525-710 psi, respectively. The empirical relationship between splitting tensile strength and compressive strength from this study is very close to that in the ACI Code and the results from Carino and Lew investigation [58]. At higher compressive strength, Equation 12.3 matches with the data better as compared with Equation 12.2. At lower compressive strength, Equation 12.3 produces lower estimated splitting tensile strength than Equation 12.2. The use of Equation 12.3 is suggested for estimating the splitting tensile strength.

The results also show that the splitting tensile strength is closely correlated to modulus of rupture. The use of splitting tensile strength to estimate the modulus of rupture is suitable.

There is no minimum requirement for splitting tensile strength in FDOT specifications. The results are close and in agreement with

studies by other researchers. It can be concluded that the splitting tensile strength of Florida concrete is in an acceptable range.

12.13.4 Modulus of Elasticity

The empirical relationship between compressive strength and modulus of elasticity from this study is also very close to that in the ACI Code. The coefficient for the ACI equation is 33 as compared to 34.1 for concrete using Brooksville aggregate at 28-day moist curing (see Equation 12.8 and Table 12.7). The use of the ACI equation to estimate the modulus of elasticity of Florida concrete (using Brooksville aggregate) is suitable.

12.13.5 Coefficient of Permeability

The coefficients of permeability of Florida Classes II, III and IV concretes are in the range of $2.5\text{--}8 \times 10^{-12}$ in/sec for Brooksville aggregate and 28-day moist curing. Concretes using Calera aggregate produced lower permeability than those using Brooksville aggregate, while river gravel exhibited slightly lower or higher permeability than Brooksville aggregate, depending on w/c-ratio and curing conditions. There are no minimum requirements for permeability of concrete in the FDOT specification. However, ACI Standard 301-72 (revised 1975) suggests that to produce watertight concrete, the w/c-ratio should be less than 0.48 and 0.44 for concrete structures that are exposed to fresh water and sea water, respectively. Neville [4] stated that a maximum permeability of 5.8×10^{-10} in/sec is recommended by some Bureau of Reclamation works.

According to the recommended requirements mentioned above, concrete using Brooksville aggregate should be suitable for use in concrete structures that are exposed to fresh or sea water.

It should be noted that all of concretes in this study are non-air-entrained concrete, and high-range water reducers were used for all batches in order to obtain the target slump of 3 inches.

CHAPTER 13 RESULTS OF FIELD TESTING PROGRAM

13.1 Introduction

The main objectives of the field testing program were to evaluate and compare the properties of in-service concrete with those of laboratory-cured concretes. The in-service concrete samples were cored samples, which were obtained from project sites at 28 days after placement. The concretes which were sampled and cast into molds at the project sites and cured in the moist room in the Department of Civil Engineering were the laboratory-cured concretes. Six projects were selected and used for this study. Table 13.1 displays the location and characteristics of these six projects. The tests, which were performed on the concrete specimens from each project, are shown in Table 4.5. The general laboratory procedure as described in Chapters 3 and 4 were used to test the concrete specimens.

The preparation for the concrete specimens for all six projects were performed by the personnel of District 2, State of Florida Department of Transportation, under the supervision of Messrs. Dan Turner, Ghulam Mujtaba, and Jack Cushman.

13.2 Test Results

The mix proportions and properties of fresh concrete for all six projects are presented in Table B.5 in Appendix B. Table B.6 displays

Table 13.1 Basic Information on the Selected Projects in the Field Testing Program

Project	FDOT Project #	Date Sampled	Concrete Class	Type of Curing	Type of Structure	Location of Project
1	26005-3518	03/03/88	IV	-	box culverts	From SR 329 to east of Seaboard Systems Railroad in Gainesville
2	33010-3518	08/05/88	IV	curing compound	pier cap #10	From SR 349 to Wideman Street in Branford
3	72020-3469	08/31/88	II	curing compound and covering with sheeting	approach slab	From Atlantic Boulevard to 0.3 mi. south-east of San Diego Road in Jacksonville
4	33010-3518	09/27/88	II	curing compound	bridge deck span #10	From SR 349 to Wideman Street in Branford
5	72001-3465	10/20/88	III	dirt and plastic	light pole footing	SR/US 9A (295)
6	72020-3479	11/04/88	III	dirt and plastic	48-inch drill shaft 8+05 Heckscher drive hole no. 8	SR-9 (I-95) at SR-105 in Jacksonville

the properties of aggregates, cement type, temperature and admixture of the concrete samples from these six projects.

The compressive strength, splitting tensile strength, modulus of rupture and coefficient of permeability of in-service concretes and laboratory-cured concretes are presented in Table C.15 in Appendix C. Tables C.16 and C.17 present the modulus of elasticity and Schmidt hammer rebound number for all six projects, respectively.

13.3 Comparison Between In-Service and Laboratory-Cured Concretes

This section presents the results of the comparison of means between the property of in-service concretes and laboratory-cured concretes for all six projects. The comparison of means is presented separately for each property of concrete.

13.3.1 Compressive Strength

A comparison of means of compressive strength was performed using Duncan's multiple-range test at a level of significance of 0.05. The results are shown in Table 13.2. The ratio of the strength of the cored samples and the strength of laboratory-cured samples are also presented in Table 13.2. It is seen that, the compressive strengths of laboratory-cured and in-service concretes are not significantly different from each other for project numbers 1, 2, 4 and 6. The compressive strength of in-service concrete is higher than that laboratory-cured concrete by about 18% for project no. 3, while for project no. 5, the in-service concrete shows lower compressive strength than the laboratory-cured concrete by about 25%.

Bloem [61] found that the compressive strengths of cored samples (4 inches in diameter) when cored from a well-cured slab are about 80% to

Table 13.2 Comparison of Mean Compressive Strength of In-Service Concretes with Laboratory-Cured Concretes at 28 Days

Project No.	Mean Compressive Strength (psi)		Comparison of Means*	$\frac{\text{Field}}{\text{Lab}} \times 100\%$
	Laboratory	Field		
1	6470	6593	NS	101.9
2	6203	6179	NS	99.6
3	4663	5518	S	118.3
4	5154	6131	NS	119.0
5	5124	3854	S	75.2
6	6514	6310	NS	96.9

Note: * Comparison of means by Duncan's test

NS = means are not significantly different at α level of 0.05

S = means are significantly different at α level of 0.05

85% of the compressive strength of 28-day laboratory-cured samples (6 inches in diameter). The possible reasons why the compressive strength of cored samples would be lower than the laboratory-cured samples are as follows:

1. The coring operation may create some damage to the cored samples
2. The consolidation of the laboratory-cured samples is more uniform
3. The laboratory-cured concretes cured more properly as compared to the in-service concretes.

However, the compressive strengths of in-service concretes are about the same or higher than those of laboratory-cured concretes for project numbers 1, 2, 3, 4 and 6. This may be due to

1. The loss of entrained air in the in-service concretes during the consolidation
2. The in-service concretes for these five projects had better quality control during the construction, curing period and coring operation.

It should also be noted that the effect of the difference in diameter of the tested samples for both concretes was not considered. Blanks and McNamara [62] found that the smaller the diameter of cylinder, the higher the compressive strength would be. A cylinder with a diameter of 4 inches shows higher compressive strength than a cylinder with a diameter of 6 inches by about 4%. The effect of the smaller diameter of cored samples compared to the larger diameter of molded cylinders has not been reported by any researchers.

13.3.2 Splitting Tensile Strength

The Duncan's multiple-range test at a level of significance of 0.05 was used to compare the means of splitting tensile strength for each project. Table 13.3 presents the results of the comparison of means and the strength ratio (by percentage). It is seen that the splitting tensile strengths of cored samples and molded cylinders are not significantly different from each other at a level of 0.05 for all six projects. The strength ratio for project no. 5 shows the highest ratio, which is different from that for the compressive strength. The low compressive strength ratio for project no. 5 might be due to the presence of microcracks in the cored samples used for the compression test resulting from the coring operations.

13.3.3 Modulus of Rupture

The modulus of rupture was not taken from the in-service concrete. The moduli of rupture of in-service concretes were estimated from the compressive strength of cored samples by using the equation suggested by ACI Code 318-83 [36]. This equation is shown below.

$$f_r = 7.5\sqrt{f'_c} \quad (13.1)$$

where f_r = modulus of rupture (psi)

f'_c = compressive strength (psi).

Table 13.4 presents the estimated moduli of rupture for all six projects.

The comparison of means of modulus of rupture was done by using Duncan's test at a level of significance of 0.05. The result of comparison of means and strength ratio are presented in Table 13.5. From

Table 13.3 Comparison of Mean Splitting Tensile Strengths of In-Service Concretes with Laboratory-Cured Concretes at 28 Days

Project No.	Mean Splitting Tensile Strength (psi)		Comparison of Means*	$\frac{\text{Field}}{\text{Lab}} \times 100\%$
	Laboratory	Field		
1	511	440	NS	86.1
2	619	587	NS	94.8
3	501	521	NS	104.0
4	497	485	NS	97.6
5	391	453	NS	115.8
6	454	454	NS	100.0

Note: * See footnotes on Table 13.2

Table 13.4 The Estimated Moduli of Rupture from the Compressive Strength of Cored Samples for All Six Projects

Project #	f'_c (psi)	$f_r = 7.5 \sqrt{f'_c}$ (psi)	Means (psi)
1	6760 6425	617 601	609
2	6553 5804	607 571	589
3	5454 5581	554 560	557
4	5955 6306	579 596	588
5	4172 3535	484 446	465
6	6083 6537	585 606	596

Table 13.5 Comparison of Mean Moduli of Rupture of Laboratory-Cured Concretes with Estimated Value from Compressive Strength of Cored Samples at 28 Days

Project No.	Mean Moduli of Rupture (psi)		Comparison of Means*	$\frac{\text{Field}}{\text{Lab}} \times 100\%$
	Laboratory	Field		
1	658	609	NS	92.6
2	669	589	NS	88.0
3	577	557	NS	96.5
4	501	588	NS	117.3
5	551	466	NS	84.6
6	590	596	NS	101.0

Note: * See footnotes on Table 13.2

Table 13.5 it can be seen that the estimated modulus of rupture from the compressive strength of in-service concretes is about the same as the modulus of rupture of laboratory-cured concretes for all six projects.

13.3.4 Coefficient of Permeability

The comparison of means of coefficient of permeability was done by using Duncan's multiple-range test at a level of significance of 0.05. The results of comparison of means and permeability ratio are presented in Table 13.6. It can be seen that the coefficients of permeability of in-service concretes are higher than those of laboratory-cured concretes for project numbers 1, 2, 3, 5 and 6, but for project no. 6 the coefficients of permeability of both concretes are not significantly different at an α level of 0.05. For project no. 4, the coefficient of permeability of in-service concrete is lower than that of laboratory-cured concrete by about 47%.

There are three possible explanations why, in general, the in-service concretes show higher values of coefficient of permeability than the laboratory-cured concretes. The first one may be due to concrete shrinkage which is caused by the variation of the temperature, humidity and wind velocity at the project sites during the first four weeks. The second explanation may be that the continued moist curing for the laboratory-cured concretes produces a higher degree of hydration than the curing compound or the plastic sheet, which was used to cure the in-service concretes. The third explanation may be that microcracks or some damage sustained during the coring operation has caused this difference in results. It can be noted that these three factors had higher effects on the permeability than on the compressive and splitting tensile strengths of the in-service concrete.

Table 13.6 Comparison of Mean Coefficients of Permeability of In-Service Concretes with Laboratory-Cured Concretes at 28 Days

Project No.	Mean Coefficient of Permeability ($\times 10^{-11}$ in/sec)		Comparison of Means*	$\frac{\text{Field}}{\text{Lab}} \times 100\%$
	Laboratory	Field		
1	4.95	6.84	S	138.2
2	1.05	2.59	S	246.7
3	7.46	11.28	S	151.2
4	8.0	4.24	S	53.0
5	5.61	10.41	S	185.6
6	4.48	6.22	NS	138.8

Note: * See footnotes on Table 13.2

For project no. 4, the in-service concretes showed lower permeability than the laboratory-cured concretes. This may be because the cored samples for this project were cored from an area that had good quality control during the construction, curing and coring operation. It can also be observed from the results of the compression test. The compressive strength of the cored sample is higher than that of the laboratory-cured sample by about 19%. The exact reason for this discrepancy is unknown.

CHAPTER 14 CONCLUSIONS AND RECOMMENDATIONS

14.1 Summary and Conclusions

This section presents the summary and conclusions of the work accomplished in this study. It includes (1) a development of a laboratory permeability test set-up, and (2) an extensive study of modulus of rupture and permeability of structural concretes in Florida.

14.1.1 Development of a Laboratory Permeability Test Set-Up

An efficient laboratory test set-up for measuring the permeability of concrete reliably has been developed and used in the laboratory testing program of this study. The developed permeability apparatus uses a cylindrical concrete specimen 4 inches in diameter and 2 inches in height. A 1-inch thick epoxy layer is cast around the side of the specimen to prevent leakage through the side. Water at a pressure of 100 psi is forced to flow from one face of the specimen to the other. The flow of water through the specimen is monitored until a steady state flow condition has been reached, and the steady state flow rate is used to calculate the coefficient of permeability of the test specimen according to Darcy's Law.

An experimental study was performed to determine the effects of specimen height on the result of the permeability test. The results of the study indicated that when the specimen height varied from 1.5 to 3 inches, the effect of specimen height was generally not significant on the results of the permeability test. Thus, confidence can be placed on

the reliability of the results of the permeability testing using a specimen height of 2 inches.

The developed test set-up has the capability to test many specimens simultaneously. The present test set-up can perform 40 permeability tests at the same time by using 40 permeability apparatuses which are connected to the same regulated compressed air source. The concrete specimens can be placed into and removed from the permeability apparatuses with a minimal loss of time and no disturbance to the other test specimens. The developed test set-up had been demonstrated to be efficient in producing consistent and reliable test results.

14.1.2 Major Findings from the Research Testing Program

An extensive study of the modulus of rupture and permeability of structural concretes in Florida has been conducted. The scope of the study covered three types of aggregate and cement, three levels of w/c-ratio, three maximum aggregate sizes, two types of pozzolan and six curing conditions. Properties of in-service concretes were evaluated and compared to those of the laboratory-cured concretes. Major findings from this study are summarized as follows:

1. The w/c-ratio is the major parameter affecting the properties of concrete. Generally, the higher the w/c-ratio is, the lower the compressive strength, modulus of rupture and splitting tensile strength and the higher the coefficient of permeability would be.
2. Curing conditions have significant effects on the properties of concrete. The 90-day moist-curing condition produces the highest compressive strength, modulus of rupture and splitting tensile strength and the lowest coefficient of permeability. The

28-day moist-curing condition produces higher compressive strength, modulus of rupture and splitting tensile strength and lower permeability than the 28-day curing with curing compound, a 3-day moist curing followed by a 25-day air curing and the 16-hour steam curing. The 16-hour steam curing produces similar effects to the 7-day moist curing, while the 28-day curing with curing compound produces similar effects to a 3-day moist curing followed by a 25-day air curing.

3. The aggregate type has some effects on the properties of concrete. Concretes using Calera aggregate generally produce higher compressive strength, splitting tensile strength and modulus of rupture, and lower permeability than those using Brooksville aggregate or river gravel. The strengths and permeability of concretes using Brooksville aggregate generally are not significantly different from those using river gravel.
4. Cement type also has some effect on the compressive strength, splitting tensile strength and modulus of rupture, especially at the early ages, while it has only a minor effect on the permeability. Type III cement produces higher strengths than Types I and II cements for concrete ages up to 28 days. The differences, however, are not significant in some cases. Concretes made with Types I, II and III cements generally exhibit about the same permeability.
5. Maximum aggregate size has no effect on the compressive strength, splitting tensile strength and modulus of rupture, while it has some effect on the permeability. The maximum aggregate size of 1 inch produces significantly higher

coefficient of permeability than the maximum aggregate sizes of $3/4$ and $3/8$ inch, which produce approximately the same permeability.

6. The fly ash concrete used in this study, which is made by replacing 20% of cement by a Class F fly ash, develops compressive strength at a relatively slower rate. At 90-day moist curing, the compressive strength of fly ash concrete is lower than that of the control concrete by about 5%.
7. The 20% fly ash replacement of cement does not produce any significant changes in the splitting tensile strength of the control tested.
8. The modulus of rupture of fly ash concrete is very close to that of the control concrete at 28 and 90 days moist curing.
9. If properly cured, the fly ash concrete generally has a lower coefficient of permeability than the control concrete.
10. The effectiveness of steam curing on fly ash concrete increases as the water-cementitious ratio decreases, as observed from the compressive and splitting tensile strength results.
11. The silica fume concrete used in this study, which is made by using silica fume as an addition, reaches its maximum compressive, splitting tensile strengths and minimum permeability at a silica fume addition of around 10% by weight of cement. It reaches its highest modulus of rupture at a silica fume addition of 20%.
12. The empirical relationships between modulus of rupture, f_r , and compressive strength, f'_c , at 28-day moist curing are as follows:

$$\text{For Brooksville aggregate, } f_r = 8.12 \sqrt{f'_c} \quad (14.1)$$

$$\text{For Calera aggregate, } f_r = 9.71 \sqrt{f'_c} \quad (14.2)$$

$$\text{For river gravel, } f_r = 9.06 \sqrt{f'_c} \quad (14.3)$$

13. The relationship between compressive strength and splitting tensile strength, f_{ct} , as expressed by $f_{ct} = f'_c{}^B$ fits the data better than the expression of $f_{ct} = A \sqrt{f'_c}$. The coefficient B for concretes using Brooksville, Calera and river gravel aggregates at 28-day moist curing are 0.712, 0.721 and 0.714, respectively.
14. The relationships between permeability and the other three strength properties of concrete are generally very poor. Table 14.1 shows the approximate ranges of coefficients of permeability corresponding to the different ranges of compressive strengths for all three aggregate types at 28-day moist curing.
15. The modulus of rupture is closely related to the splitting tensile strength. The following equations are the equations obtained from this study (at 28-day moist curing):

$$\text{For Brooksville aggregate, } f_{ct} = 0.83 f_r \quad (14.4)$$

$$\text{For Calera aggregate, } f_{ct} = 0.78 f_r \quad (14.5)$$

$$\text{For river gravel, } f_{ct} = 0.76 f_r \quad (14.6)$$

16. The empirical relationships between compressive strength and modulus of elasticity of concrete specimens at 28-day moist curing are as follows:

Table 14.1 Approximate Ranges of Coefficient of Permeability
as Compared to Compressive Strength of Concrete at
28-Day Moist Curing

Range of Compressive Strength (psi)	Range of Coefficient of Permeability ($\times 10^{-12}$ in/sec)		
	Brooksville Aggregate	River Gravel	Calera Aggregate
7000-8000	4.5-8.0	3.0-7.0	-
8000-9000	3.0-7.0	2.5-5.0	2.0-4.0
> 9000	2.4-4.0	2.0-3.0	1.0-3.0

$$\text{For Brooksville aggregate} \quad E = 34.05 w^{1.5} \sqrt{f'_c} \quad (14.7)$$

$$\text{For Calera aggregate,} \quad E = 40.31 w^{1.5} \sqrt{f'_c} \quad (14.8)$$

$$\text{For river gravel,} \quad E = 34.23 w^{1.5} \sqrt{f'_c} \quad (14.9)$$

The averaged unit weight of concrete made with Brooksville, Calera and river gravel aggregates are 145, 152 and 150 pcf, respectively.

17. The compressive strength and splitting tensile strength of in-service concrete and laboratory-cured concrete generally are about the same. The modulus of rupture that is estimated from the compressive strength of in-service concrete by using the ACI equation is not significantly different from the measured modulus of rupture of the laboratory-cured concrete. The coefficient of permeability of in-service concrete is generally substantially higher than that of the laboratory-cured concrete.

14.2 Recommendations

The findings drawn from this study lead to the following recommendations:

1. The properties of concrete are affected by the mix parameters and curing condition. Not only appropriate mix proportioning but also adequate moist curing should be required for concrete projects where strength and low permeability are important. Air curing or curing by use of curing compound will produce concrete of relatively poorer quality.

2. The Brooksville aggregate can be used to produce high strengths and low permeability concretes. It is recommended that it be used for structural concrete.
3. Fly ash can be used to improve the long-term strength and reduce the permeability of control concrete, especially for concrete with high w/c-ratio. However, proper curing is needed in order to achieve this goal.
4. Results from this study show that silica fume concrete made with Brooksville aggregate exhibit higher strengths and lower permeability as compared with control concrete with the same condition. Therefore, silica fume concrete is strongly recommended for concrete structures that require high strength and low permeability. An addition of 10% of silica fume by weight of cement is recommended as the optimum amount to be used.
5. Results from this study show that the relationships between permeability and the other properties is very poor. A direct measurement of concrete permeability should be made for control of this property.
6. The splitting tensile strength can be used to estimate the modulus of rupture of concrete.

14.3 Research Needs

The following are present research needs in the area of durability of concrete used in Florida.

1. The laboratory testing program of this study should be extended to structural concretes made with other types of Florida aggregates (besides Brooksville aggregate) to generate a broader

database for the coefficient of permeability and modulus of rupture of structural concretes in Florida.

2. The optimum replacement of cement by fly ash to obtain the lowest coefficient of permeability and the highest modulus of rupture, should be investigated.
3. The effect of the added silica fume to the fly ash concrete should be studied to determine the optimum combination of fly ash and silica fume to be used.
4. The relationship between chloride permeability and water permeability and the relationship between corrosion protection and water permeability of concrete should be investigated. The results from this study can be used to develop criteria for selection of concrete mixes to achieve improved durability.
5. The effect of the use of superplasticizer on concrete durability should be investigated.
6. The effect of air-entraining admixture on concrete permeability should also be studied.
7. The permeability of the aggregate and that of the cement paste should be determined separately. The result from this study will lead to better understanding of the effect of the aggregate type on concrete permeability.
8. Blast furnace slag has also been used to replace cement in structural concretes in Florida. Therefore, a detailed study on the effects of blast furnace slag on the modulus of rupture and coefficient of permeability of Florida concrete should also be investigated.

APPENDIX A
MIX PROPORTIONS AND PROPERTIES OF TRIAL BATCHES

Table A.1 Mix Proportions of Trial Batches

Trial Batch No.	Concrete Class	W/C-Ratio	Aggregate Type	Cement (lb/yd ³)	Pozzolan (lb/yd ³)	Water (lb/yd ³)	Sand (lb/yd ³)	Coarse Aggregate (lb/yd ³)	Admixture RD-1 (lb/yd ³)
1	II	0.45	Brooksville #67	564	0	254	1286	1702	2.2
2	II	0.45	Brooksville #89	564	0	254	1688	1304	3.9
3	II	0.45	Brooksville #57	564	0	254	1180	1806	1.6
4	II	0.45	Calera #67	564	0	254	1383	1812	2.1
5	II	0.45	River Gravel #67	564	0	254	1230	1887	2.2
6	II with fly ash	0.45	Brooksville #67	451	113	254	1286	1702	3.2
7	II with silica fume	0.45	Brooksville #67	451	113	254	1286	1702	3.2
8	III	0.38	Brooksville #67	658	0	250	1220	1702	3.2
9	IV	0.33	Brooksville #67	752	0	248	1149	1702	4.9

Note: Sand and coarse aggregate are dry weight

Table A.2 Properties of Trial Batches

Trial Batch No.	Slump (inches)	Air Content (Percent)	Unit Weight (lb/ft ³)	7-Day Mean Strength (psi)		28-Day Mean Strength (psi)	
				Compressive	Tensile	Compressive	Tensile Flexural
1*	3.25	-	146	4275	415	5391	589 978
2	2.75	3.0	145	4476	377	7046	457 939
3	2.75	3.5	147	3935	368	5518	472 915
4	3.25	2.5	152	5363	511	6418	496 1071
5	2.75	2.5	150	4503	466	5669	499 602
6	3.0	-	145	3898	361	5224	592 607
7	3.0	1.75	145	5393	500	7361	621 709
8	3.0	3.0	146	5443	421	6535	664 1017
9	2.75	2.5	147	4965	507	7553	508 939

Note: * See Table A.1 for identifications of the trial batches

APPENDIX B
MIX PROPORTIONS AND PROPERTIES OF FRESH CONCRETE
IN MAIN LABORATORY AND FIELD TESTING PROGRAM

Table B.1 Mix Proportions of Concrete Mixes in the Main Laboratory Testing Program

Batch No.	Concrete Class	Aggregate Type	W/C Ratio	Cement Type	Cement (lb/yd ³)	Water (lb/yd ³)	Sand (lb/yd ³)	Coarse Aggregate (lb/yd ³)	Admixture RD-1 (lb/yd ³)
1	II	Brooksville #67	0.45	II	564	254	1286	1702	6.0
2	III	Brooksville #67	0.38	I	658	250	1220	1702	4.9
3	II	Brooksville #67	0.45	I	564	254	1286	1702	6.2
4	III	Brooksville #67	0.38	II	658	250	1220	1702	5.8
5	IV	Brooksville #67	0.33	III	752	248	1149	1702	13.4
6	IV	Brooksville #67	0.33	II	752	248	1149	1702	6.8
7	IV	Brooksville #89	0.33	I	752	248	1551	1304	7.6
8	II	River Gravel #67	0.45	I	564	254	1230	1886	5.6
9	II	Calera #67	0.45	I	564	254	1418	1757	6.3
10	IV	Brooksville #67	0.33	I	752	248	1190	1649	7.2

Table B.1--continued

Batch No.	Concrete Class	Aggregate Type	W/C Ratio	Cement Type	Cement (lb/yd ³)	Water (lb/yd ³)	Sand (lb/yd ³)	Coarse Aggregate (lb/yd ³)	Admixture RD-1 (lb/yd ³)
11	IV	Brooksville #89	0.33	I	752	248	1587	1252	7.8
12	IV	Brooksville #57	0.33	I	752	248	1085	1754	6.5
13	IV	Calera #57	0.33	II	752	248	1283	1757	5.3
14	III	Brooksville #67	0.38	III	658	250	1234	1679	6.1
15	IV	Calera #67	0.33	III	752	248	1263	1789	6.8
16	II	Brooksville #67	0.45	III	564	254	1305	1679	4.8
17	IV	River Gravel #67	0.33	II	752	248	1113	1862	4.6
18	III	Calera #67	0.38	I	658	250	1334	1789	5.6
19	II	Brooksville #67	0.45	II + Fly Ash	451+113	254	1305	1679	4.4
20	III	River Gravel #67	0.38	I	658	250	1183	1862	4.3

Table B.1--continued

Batch No.	Concrete Class	Aggregate Type	W/C Ratio	Cement Type	Cement (lb/yd ³)	Water (lb/yd ³)	Sand (lb/yd ³)	Coarse Aggregate (lb/yd ³)	Admixture RD-1 (lb/yd ³)
21	IV	Calera #67	0.33	I	752	248	1263	1789	6.1
22	II	Brooksville #67	0.45	III	564	254	1323	1657	8.1
23	IV	River Gravel #67	0.33	III	752	248	1133	1837	7.5
24	IV	Brooksville #67	0.33	II + Fly Ash	602+150	248	1187	1657	6.7
25	IV	River Gravel #67	0.33	I	752	248	1133	1837	5.2
26	III	Brooksville #67	0.38	II + Fly Ash	526+132	250	1257	1657	5.0
27	III	Calera #67	0.38	I	658	250	1351	1765	6.2
28	IV	Brooksville #67	0.33	III	752	248	1187	1657	9.3
29	II	Brooksville #67	0.45	I	564	254	1323	1657	5.7
30	IV	River Gravel #67	0.33	III	752	248	1114	1856	8.9

Table B.1--continued

Batch No.	Concrete Class	Aggregate Type	W/C Ratio	Cement Type	Cement (lb/yd ³)	Water (lb/yd ³)	Sand (lb/yd ³)	Coarse Aggregate (lb/yd ³)	Admixture RD-1 (lb/yd ³)
31	II	River Gravel #67	0.45	I	564	254	1250	1856	4.0
32	III	Brooksville #67	0.38	II + Fly Ash	526+132	250	1239	1674	7.9
33	II	Calera #67	0.45	I	564	254	1399	1783	4.9
34	IV	Brooksville #57	0.33	I	752	248	1087	1776	6.1
35	IV	River Gravel #67	0.33	II	752	248	1139	1853	4.6
36	III	Brooksville #67	0.38	III	658	250	1266	1672	8.5
37	IV	Calera #67	0.33	III	752	248	1291	1781	10.4
38	IV	Calera #67	0.33	I	752	248	1323	1778	6.3
39	II	Brooksville #67	0.45	II	564	254	1367	1669	5.1
40	IV	Calera #67	0.33	II	752	248	1290	1850	5.6

Table B.1--continued

Batch No.	Concrete Class	Aggregate Type	W/C Ratio	Cement Type	Cement (lb/yd ³)	Water (lb/yd ³)	Sand (lb/yd ³)	Coarse Aggregate (lb/yd ³)	Admixture RD-1 (lb/yd ³)
41	IV	Brooksville #67	0.33	I	752	248	1275	1597	7.5
42	III	Brooksville #67	0.38	I	658	250	1349	1597	6.7
43	III	River Gravel #67	0.38	I	658	250	1242	1851	5.8
44	IV	Brooksville #67	0.33	II	752	248	1275	1597	5.8
45	III	Brooksville #67	0.38	II	658	250	1349	1597	5.2
46	IV	Brooksville #67	0.33	II + Fly Ash	602+150	248	1275	1597	7.3
47	IV	River Gravel #67	0.33	I	752	248	1165	1851	7.5
48	II	Brooksville #67	0.45	II + Fly Ash	451+113	254	1401	1597	4.8

Note: (see next page)

Table B.1--continued

<u>Note:</u>	Sand and coarse aggregate are dry weight
	Batch No. 1-8 used sand from truck 1
	Batch No. 9-13 used sand from truck 2
	Batch No. 14-21 used sand from truck 3
	Batch No. 22-33 used sand from truck 4
	Batch No. 34-37 used sand from truck 5
	Batch No. 38-46 used sand from truck 6
	Batch No. 47-48 used sand from truck 7
-	Coarse aggregate (Brooksville #67) for Batch Nos. 41, 42, 44, 45, 46, 48 is different from the previous batches
-	Coarse aggregate (Calera) for Batch No. 40 is different from the previous batches
-	Coarse aggregate (river gravel) for Batch No. 47 is different from the previous batches

Table B.2 Properties of Fresh Concrete in the
Main Laboratory Testing Program

Batch No.	Slump (inches)	Air Content (Percent)	Unit Weight (lb/ft ³)
1	2.75	3.0	145
2	3.25	2.25	146
3	2.88	3.25	144
4	2.75	2.25	146
5	3.25	2.0	148
6	3.25	2.25	148
7	2.75	3.0	144
8	3.50	4.0	149
9	3.25	2.75	152
10	3.25	2.25	147
11	2.75	2.5	145
12	3.25	2.5	147
13	3.0	2.5	152
14	2.75	2.0	147
15	3.25	2.25	153
16	3.5	2.75	145
17	3.5	2.75	151
18	3.25	2.25	153
19	3.0	2.25	145
20	2.5	2.5	151
21	3.0	2.5	152
22	3.25	2.5	145
23	2.5	2.75	151
24	3.0	2.5	145

Table B.2--continued

Batch No.	Slump (inches)	Air Content (Percent)	Unit Weight (lb/ft ³)
25	2.5	1.75	151
26	3.5	2.5	145
27	2.62	2.75	152
28	3.0	1.75	146
29	2.62	2.25	146
30	3.25	1.75	151
31	3.0	2.5	150
32	2.75	2.75	147
33	3.25	2.25	153
34	2.5	2.5	148
35	3.0	2.5	149
36	2.75	2.75	148
37	2.5	2.75	153
38	3.25	2.5	152
39	2.5	2.5	144
40	3.0	2.25	153
41	3.0	2.25	146
42	3.25	2.25	146
43	3.5	2.0	148
44	3.25	2.5	146
45	3.0	2.5	147
46	2.75	2.25	147
47	2.75	2.25	151
48	3.0	2.5	145

Table B.3 Mix Proportions of Silica Fume Concrete

Batch No.	Silica Fume (Percent)	Silica Fume (lb/yd ³)	Cement (lb/yd ³)	Water (lb/yd ³)	Sand (lb/yd ³)	Coarse Aggregate (lb/yd ³)	Admixture RD-1 (lb/yd ³)
S1	5	38	752	261	1107	1674	5.2
S2	5	38	752	261	1107	1674	4.9
S3	10	75	752	273	1022	1674	4.9
S4	10	75	752	273	1022	1674	5.0
S5	15	113	752	285	947	1674	2.4
S6	15	113	752	285	969	1672	2.1
S7	20	150	752	298	894	1672	1.6
S8	20	150	752	298	894	1672	1.2
S9	10	75	752	273	1140	1851	1.6
S10	20	150	752	298	985	1851	1.2

Note:

- Concrete Class IV
- Water-Cementitious ratio = 0.33
- Cement Type = Type II
- Batch No. S1-S8 used Brooksville #67
- Batch No. S9-S10 used Calera #67
- Sand and coarse aggregate are dry weight
- Batch No. S1-S5 used sand from truck 4
- Batch No. S6-S8 used sand from truck 5
- Batch No. S9-S10 used sand from truck 6

Table B.4 Properties of Fresh Concrete for Silica Fume Concrete

Batch No.	Slump (inches)	Air Content (Percent)	Unit Weight (lb/ft ³)
S1	3.25	2.0	146
S2	3.0	2.25	147
S3	2.75	2.0	146
S4	2.62	2.0	144
S5	2.75	2.0	146
S6	2.75	1.75	144
S7	2.75	2.25	146
S8	2.5	2.25	145
S9	2.5	2.0	154
S10	2.5	2.0	153

Table B.5 Mix Proportions and Properties of Fresh Concrete in Field Testing Program

Project No.	Concrete Class	W/C Ratio	Mix Proportions (lb/yd ³)				Properties of Fresh Concrete		
			Coarse Aggregate	Fine Aggregate	Water	Cement	Slump (inches)	Air Content (Percent)	Unit Weight (lb/ft ³)
1	IV	0.40	1630	1116	275	690	2.25	3.5	143.6
2	IV	0.37	1820	1022	247	658	2.75	5.5	142.2
3	II	0.44	1723	1191	256	564	5.5	6.0	137.1
4	II	0.41	1741	1258	169	564	5.0	5.0	138.7
5	III	0.42	1470	1300	217	705	8.0	2.5	140.1
6	III	0.36	1480	1300	175	705	8.5	2.5	140.1

Table B.6 Properties of Aggregate, Cement Type, Temperature and Admixture in Field Testing Program

Project No.	Cement Type	Coarse Aggregate			Fine Aggregate			Temperature (°F)			Admixture	
		S.G (SSD)	Pit No.	Fineness Modulus	S.G (SSD)	Pit No.	Air	Concrete	Entrainment	Other		
1	I	57	2.36	34-196 (Gulf Hammock)	2.43	2.63	76-137 Keuka	72	79	Sika AER	Plastocrete-161 MR	
2	I	57	2.44	38-036 (Cabbage Grove)	2.12	2.64	GA-303 (Mayday)	-	90	Darex	WRDA 79	
3	II	57	2.43	87-090 (F.E.C. Quarry)	2.30	2.60	76-349 (Grandin)	84	91	Darex	WRDA 79	
4	I	57	2.44	38-036 (Cabbage Grove)	2.12	2.64	GA-303 (Mayday)	72	82	Darex	WRDA 79	
5	II	57	2.43	87-049 (Golden Prince Quarry)	2.30	2.60	76-349 (Grandin)	-	79	Darex	WRDA 79	
6	I	57	2.43	87-049 (Golden Prince Quarry)	2.30	2.6	76-349 (Grandin)	-	82	Darex	WRDA 79	

APPENDIX C
SUMMARY OF TEST RESULTS IN MAIN LABORATORY
AND FIELD TESTING PROGRAM

Table C.1 Modulus of Rupture of Concrete in the Main Laboratory Testing Program

Batch No.	Specimen No.	Curing Condition*					
		1	2	3	4	5	6
1	1	596	590	543	590	602	661
	2	602	602	543	590	602	657
2	1	674	602	601	590	584	672
	2	662	590	618	571	567	648
3	1	614	590	669	532	673	680
	2	638	578	662	520	626	654
4	1	672	693	679	472	656	704
	2	690	709	654	520	569	686
5	1	806	833	873	823	839	884
	2	823	778	856	822	850	893
6	1	638	540	711	632	645	792
	2	637	555	705	642	640	752
7	1	664	773	774	672	675	796
	2	639	763	800	661	676	721
8	1	530	543	662	625	620	723
	2	547	549	680	596	615	738
9	1	676	719	779	636	608	945
	2	658	734	743	745	621	924
10	1	674	760	798	683	660	802
	2	684	802	819	701	726	773
11	1	729	717	811	631	661	787
	2	701	741	788	634	652	825
12	1	615	695	702	596	632	720
	2	598	696	774	631	575	785
13	1	887	873	893	715	689	965
	2	864	878	920	705	720	1020

Table C.1--continued

Batch No.	Specimen No.	Curing Condition*					
		1	2	3	4	5	6
14	1	779	793	779	614	650	778
	2	821	811	820	655	618	804
15	1	1123	1056	1056	717	695	1070
	2	1018	989	1089	737	725	1086
16	1	660	760	715	575	581	770
	2	690	744	740	598	583	745
17	1	637	719	744	551	636	823
	2	643	749	821	563	587	945
18	1	778	899	876	752	778	918
	2	770	853	892	735	716	971
19	1	513	542	596	579	634	646
	2	448	545	651	631	587	680
20	1	639	692	778	609	620	864
	2	634	708	738	693	750	754
21	1	904	952	959	743	691	1079
	2	816	875	923	796	764	1106
22	1	640	732	717	562	538	786
	2	649	778	733	537	573	729
23	1	743	910	998	637	658	976
	2	784	905	887	692	683	1079
24	1	579	678	679	483	521	702
	2	580	687	707	569	566	733
25	1	704	691	939	749	662	855
	2	695	725	816	684	780	834
26	1	536	573	633	610	560	670
	2	471	557	602	519	567	697

Table C.1--continued

Batch No.	Specimen No.	Curing Condition*					
		1	2	3	4	5	6
27	1	763	834	867	639	608	908
	2	821	849	853	705	650	995
28	1	847	828	881	715	703	823
	2	815	850	847	648	686	868
29	1	605	681	679	596	595	685
	2	611	673	740	569	609	733
30	1	915	943	1058	696	711	1115
	2	897	933	1017	863	767	1197
31	1	651	628	663	574	689	719
	2	551	624	698	674	583	763
32	1	573	609	645	554	610	817
	2	591	589	691	531	577	826
33	1	757	796	734	625	707	932
	2	761	782	867	630	637	890
34	1	631	717	805	657	620	772
	2	695	723	737	618	687	811
35	1	673	698	847	685	656	1071
	2	660	728	923	620	608	992
36	1	691	739	740	596	593	815
	2	680	763	780	560	562	865
37	1	951	1077	1091	792	855	1186
	2	964	1100	1107	884	780	1314
38	1	794	976	953	799	744	1188
	2	832	955	1033	849	800	1103
39	1	632	637	707	593	614	719
	2	661	684	737	633	663	755

Table C.1--continued

Batch No.	Specimen No.	Curing Condition*					
		1	2	3	4	5	6
40	1	737	952	1020	679	648	1129
	2	772	965	981	656	684	1075
41	1	687	744	829	645	639	809
	2	681	808	839	693	633	853
42	1	674	627	749	626	602	790
	2	689	732	729	603	658	768
43	1	637	715	779	595	560	873
	2	655	725	781	656	620	900
44	1	701	732	768	652	636	767
	2	639	695	755	586	595	805
45	1	625	714	744	495	504	741
	2	738	687	670	553	507	768
46	1	638	737	713	475	502	738
	2	644	725	750	525	467	779
47	1	756	772	887	732	646	988
	2	683	790	893	622	685	956
48	1	625	637	613	530	475	782
	2	599	598	638	543	561	713

Note: * Curing conditions:

1 = 16 hours steam curing

2 = 7 days moist curing

3 = 28 days moist curing

4 = 3 days moist curing and 25 days in air

5 = 1 day in form and 27 days with curing compound

6 = 90 days moist curing

Table C.2 Compressive Strengths of Concrete in the Main Laboratory Testing Program

Batch No.	Specimen No.	Curing Condition* (psi)					
		1	2	3	4	5	6
1	1	4860	5129	7011	6519	6268	7984
	2	4461	5341	7216	6629	5695	7913
2	1	5458	6052	7382	7948	7499	8447
	2	5564	6038	7354	7559	7630	8185
3	1	5073	5702	7605	7160	7273	7973
	2	5111	5773	7400	7160	7252	8295
4	1	5172	6119	7623	7043	7308	8606
	2	5246	6226	7899	7375	7142	8702
5	1	8412	8712	10340	10127	9876	10570
	2	8815	8451	10524	9912	9936	10265
6	1	6838	6668	8302	8097	8242	9646
	2	6629	6491	8861	7503	8150	9583
7	1	6778	6915	9289	8811	8939	9795
	2	6908	7283	9031	8932	8380	9710
8	1	5264	5069	7198	7018	6700	7860
	2	5447	5377	7605	6763	6707	7634
9	1	5982	5918	7860	7397	6615	8291
	2	5844	5989	7570	7032	6816	7796
10	1	6735	6891	8861	8691	8847	9777
	2	6848	6792	8939	8985	8833	9519
11	1	6898	6700	9402	8412	8939	9993
	2	6859	7106	9094	8815	8394	10099
12	1	6491	6381	8228	8005	7711	8659
	2	6272	6608	8069	7446	7715	9020
13	1	7446	7382	9278	8447	8323	10909
	2	7740	7545	9324	8458	8115	10722

Table C.2--continued

Batch No.	Specimen No.	Curing Condition* (psi)					
		1	2	3	4	5	6
14	1	6937	7064	8313	7938	7863	8946
	2	6671	7018	8543	8016	8001	8507
15	1	8203	8033	9480	8946	8465	10251
	2	8277	7987	9091	8921	8447	10318
16	1	5677	5653	7269	6901	6724	7297
	2	5423	5745	7167	6770	6763	7156
17	1	5904	5826	8008	7902	7103	8794
	2	5716	5706	7906	7510	6852	9006
18	1	5932	6696	8702	8313	8348	9929
	2	6158	6523	8765	8284	8076	9561
19	1	3364	3891	6102	5720	5416	7156
	2	3626	3994	6134	5571	5387	7131
20	1	6120	6091	8221	8157	7662	8348
	2	5989	6141	8076	7863	7704	8001
21	1	7729	7616	10053	9317	9749	11507
	2	7559	7687	9798	9749	9445	10768
22	1	6434	6233	7658	7489	7386	7694
	2	6463	6304	7531	7206	7658	7570
23	1	8100	7588	9883	9505	9590	10343
	2	8168	8649	9650	9462	9413	9908
24	1	6250	5787	7878	7793	7344	8210
	2	6367	5960	7902	7860	7588	8164
25	1	7425	7106	8475	8291	8253	9321
	2	7181	6533	8758	8684	8571	9912
26	1	4103	3813	6240	5858	5904	6873
	2	4047	3859	6360	6137	5727	6919

Table C.2--continued

Batch No.	Specimen No.	Curing Condition* (psi)					
		1	2	3	4	5	6
27	1	6491	6721	8758	8589	8150	10361
	2	6349	6944	9080	8475	8553	10559
28	1	8398	8440	9862	9416	9650	10064
	2	8380	7899	9614	9767	9685	9830
29	1	5271	5554	7591	7290	7354	7931
	2	5175	5621	7414	7411	7612	7913
30	1	8666	8744	10081	10057	10145	10548
	2	8429	8776	9965	9919	9859	10771
31	1	4956	5225	6880	6820	6208	7757
	2	4952	5186	6869	6615	5692	7612
32	1	5090	5819	6947	6562	6753	8383
	2	5324	5964	7170	6495	6512	8281
33	1	6325	6767	8826	8458	7888	9622
	2	6240	6802	8967	8504	8171	9678
34	1	6912	6997	8560	8447	8214	9466
	2	7036	6873	8553	8426	8207	9119
35	1	6915	6813	9335	9063	8362	10446
	2	6923	6976	9268	9342	8521	9954
36	1	7803	8037	8773	8857	8734	8794
	2	7517	7945	8882	8543	8628	9105
37	1	9522	10011	10930	10753	10796	11341
	2	9448	9735	11005	10930	10180	11401
38	1	7570	8822	10081	9622	9052	11252
	2	7672	8200	9823	9576	9300	10941
39	1	4821	5486	7439	6976	7082	7612
	2	4577	5656	7386	6827	7167	7825

Table C.2--continued

Batch No.	Specimen No.	Curing Condition* (psi)					
		1	2	3	4	5	6
40	1	7301	7474	9169	8507	8001	9724
	2	7503	7110	8974	8557	8069	10375
41	1	6682	7343	8988	8723	8638	9087
	2	6710	7443	8942	8642	8872	9377
42	1	5950	6385	7771	7761	7757	8249
	2	5621	6671	7846	7545	7672	8415
43	1	6484	6548	8309	7935	8037	8352
	2	6434	6788	8182	7761	8019	8571
44	1	6516	6859	8245	8461	8210	8649
	2	6148	6604	8429	8171	8005	8444
45	1	5706	5960	7506	7276	7266	7662
	2	5897	6067	7379	7453	7552	7591
46	1	6785	6717	8708	8100	7803	9211
	2	6629	7053	8748	7998	7991	9158
47	1	7177	7276	8751	8709	8451	9285
	2	7244	7336	8359	8822	8270	9133
48	1	4213	4662	6304	5964	5469	7538
	2	4337	4648	6279	6180	5709	7867

Note: * See footnotes on Table C.1

Table C.3 Splitting Tensile Strengths of Concrete
in the Main Laboratory Testing Program

Batch No.	Specimen No.	Curing Condition*					
		1	2	3	4	5	6
1	1	253	323	450	483	546	587
	2	251	303	425	483	510	529
2	1	485	599	451	515	501	556
	2	523	552	432	439	544	600
3	1	412	414	558	527	495	582
	2	361	511	554	515	564	585
4	1	366	449	620	511	602	657
	2	404	472	615	443	569	679
5	1	679	659	728	663	678	726
	2	640	641	697	676	678	721
6	1	496	487	657	599	628	705
	2	517	467	616	525	595	640
7	1	582	638	725	643	631	719
	2	639	674	698	663	679	709
8	1	490	509	527	511	470	586
	2	519	487	555	481	513	606
9	1	528	547	588	502	497	678
	2	548	567	587	503	532	690
10	1	639	600	766	496	565	717
	2	630	648	658	556	550	771
11	1	629	618	648	585	603	740
	2	619	655	698	552	567	714
12	1	544	561	686	588	592	702
	2	554	553	585	495	481	667
13	1	687	624	775	653	642	948
	2	694	658	781	593	683	847

Table C.3--continued

Batch No.	Specimen No.	Curing Condition*					
		1	2	3	4	5	6
14	1	640	610	591	541	596	641
	2	593	617	632	568	558	710
15	1	727	761	745	619	660	846
	2	686	656	717	687	638	769
16	1	436	495	550	499	492	680
	2	452	524	518	478	507	618
17	1	571	585	658	648	557	618
	2	525	538	640	601	596	690
18	1	610	611	701	718	712	794
	2	578	640	709	677	666	758
19	1	382	407	518	453	439	581
	2	364	473	548	410	425	586
20	1	464	542	656	628	617	681
	2	520	468	674	686	634	677
21	1	663	646	885	722	667	851
	2	683	723	775	771	722	851
22	1	511	551	649	592	576	661
	2	559	546	569	505	509	576
23	1	581	619	678	676	614	760
	2	664	649	627	620	655	695
24	1	523	499	528	519	509	732
	2	552	500	572	510	562	643
25	1	527	502	658	612	582	716
	2	525	515	640	606	584	617
26	1	474	451	576	538	453	640
	2	473	461	548	489	441	611

Table C.3--continued

Batch No.	Specimen No.	Curing Condition*					
		1	2	3	4	5	6
27	1	654	710	776	672	667	806
	2	570	674	686	579	593	744
28	1	643	663	692	642	610	662
	2	598	606	709	624	638	686
29	1	584	523	605	563	616	615
	2	579	519	559	548	494	581
30	1	647	637	714	619	648	759
	2	674	693	707	617	692	794
31	1	403	459	599	550	472	658
	2	461	432	560	550	525	614
32	1	437	510	600	534	518	661
	2	429	509	603	502	589	644
33	1	480	579	616	658	643	790
	2	554	552	634	611	598	735
34	1	596	573	673	609	636	674
	2	578	601	613	618	605	660
35	1	613	536	736	655	643	732
	2	640	566	653	668	664	624
36	1	577	564	586	544	573	679
	2	581	547	611	600	620	630
37	1	660	703	819	735	809	839
	2	686	816	775	786	750	836
38	1	640	722	846	745	740	833
	2	696	738	794	785	680	898
39	1	410	461	577	524	546	602
	2	391	459	590	505	515	576

Table C.3--continued

Batch No.	Specimen No.	Curing Condition*					
		1	2	3	4	5	6
40	1	661	663	727	594	602	810
	2	629	600	706	652	595	751
41	1	635	629	728	558	569	676
	2	603	610	677	568	595	772
42	1	497	591	575	558	533	739
	2	476	529	586	569	563	773
43	1	528	505	653	622	669	657
	2	524	499	648	680	596	674
44	1	483	504	629	610	633	675
	2	502	497	612	541	636	759
45	1	501	447	556	510	547	620
	2	448	506	565	521	462	571
46	1	494	501	662	616	631	718
	2	561	547	667	621	570	652
47	1	514	534	625	711	617	763
	2	538	570	617	614	628	748
48	1	377	424	543	521	414	649
	2	411	475	510	467	517	587

Note: * See footnotes on Table C.1

Table C.4 Coefficient of Permeability of Concrete
in the Main Laboratory Testing Program

Batch No.	Specimen No.	Curing Condition* ($\times 10^{-12}$ in/sec)				
		1	3	4	5	6
1	1	20.67	6.72	7.51	7.70	6.72
	2	18.48	7.12	8.49	7.51	6.92
2	1	7.72	4.18	6.59	6.33	4.73
	2	6.65	4.81	5.55	5.71	4.98
3	1	12.05	5.35	5.55	6.04	5.35
	2	13.97	5.88	7.12	6.33	5.63
4	1	8.29	4.57	4.96	5.94	3.40
	2	8.88	4.38	4.77	4.96	3.94
5	1	3.32	2.22	3.20	3.01	0.93
	2	2.89	2.53	3.01	4.01	1.31
6	1	3.59	2.81	3.98	3.89	2.87
	2	3.40	3.19	3.79	3.40	2.63
7	1	5.03	3.49	3.79	4.96	2.81
	2	5.54	4.18	4.37	4.77	1.95
8	1	18.67	6.33	8.31	8.86	6.14
	2	16.71	7.70	8.13	8.49	5.94
9	1	6.63	2.22	3.00	3.45	2.06
	2	5.49	2.42	3.22	2.61	2.22
10	1	6.55	3.08	3.83	4.57	2.81
	2	7.20	2.89	3.59	4.73	3.20
11	1	5.51	2.77	3.34	3.44	2.39
	2	4.47	2.72	3.02	3.49	2.43
12	1	14.60	7.67	8.37	8.12	5.62
	2	14.56	6.76	9.27	7.90	4.60
13	1	2.46	1.35	2.29	2.86	1.27
	2	2.74	1.18	3.01	2.27	1.23

Table C.4--continued

Batch No.	Specimen No.	Curing Condition* ($\times 10^{-12}$ in/sec)				
		1	3	4	5	6
14	1	11.62	4.51	6.72	5.75	4.08
	2	11.95	4.96	7.51	7.12	4.22
15	1	1.88	1.30	2.12	2.04	1.11
	2	2.27	1.43	2.36	1.79	1.09
16	1	13.91	6.17	6.37	6.83	5.39
	2	14.51	5.93	6.52	6.42	5.13
17	1	8.15	2.84	3.88	4.55	2.52
	2	6.72	2.84	3.32	5.04	2.37
18	1	2.98	2.03	2.18	2.42	1.87
	2	2.82	1.90	2.46	2.27	1.93
19	1	12.38	3.23	11.73	12.60	2.91
	2	15.12	4.70	11.02	13.97	2.87
20	1	6.20	3.94	5.44	5.13	3.29
	2	6.59	3.29	4.76	4.76	2.94
21	1	4.26	1.59	1.83	2.51	1.55
	2	3.74	1.68	2.20	2.28	1.47
22	1	14.57	5.93	5.68	9.13	5.96
	2	14.38	6.29	7.34	6.89	5.60
23	1	4.81	2.21	2.51	2.53	1.92
	2	4.27	1.98	2.51	2.51	1.41
24	1	2.82	2.84	3.58	3.56	2.0
	2	3.41	2.59	4.16	3.80	1.65
25	1	3.45	2.37	3.76	2.57	1.67
	2	3.25	2.45	2.74	2.67	1.54
26	1	3.49	4.33	4.58	4.72	3.85
	2	4.51	4.03	5.11	5.17	3.83

Table C.4--continued

Batch No.	Specimen No.	Curing Condition* (x 10 ⁻¹² in/sec)				
		1	3	4	5	6
27	1	2.19	2.06	3.06	3.12	2.00
	2	2.39	2.74	3.16	3.20	1.83
28	1	3.56	2.88	3.23	3.40	2.11
	2	3.02	2.74	3.22	3.23	2.01
29	1	14.59	4.17	5.98	5.50	4.59
	2	13.83	5.03	5.04	5.96	4.75
30	1	3.05	1.78	3.29	2.33	1.47
	2	3.06	2.00	2.43	3.62	1.25
31	1	12.97	5.14	5.72	7.38	4.27
	2	12.81	5.26	5.98	7.89	5.37
32	1	3.92	3.53	4.75	4.23	1.81
	2	3.43	3.45	4.22	3.87	2.60
33	1	4.24	3.51	4.17	5.50	3.41
	2	4.11	3.98	3.35	4.17	3.90
34	1	11.25	3.57	5.58	4.62	3.37
	2	10.33	3.11	5.41	4.80	4.35
35	1	5.44	2.76	3.43	3.80	2.59
	2	4.81	2.70	4.25	4.07	2.65
36	1	6.16	4.31	6.01	5.60	5.12
	2	5.55	5.08	7.31	6.05	4.88
37	1	2.75	2.12	2.66	3.01	1.19
	2	2.67	2.72	2.16	2.99	1.30
38	1	3.40	1.93	3.77	2.32	1.47
	2	3.91	2.15	2.41	3.09	1.59
39	1	29.04	7.69	7.35	7.57	6.22
	2	31.78	7.05	7.82	7.49	5.55

Table C.4--continued

Batch No.	Specimen No.	Curing Condition* ($\times 10^{-12}$ in/sec)				
		1	3	4	5	6
40	1	3.01	3.07	3.92	4.46	1.98
	2	3.22	2.93	3.41	4.42	2.41
41	1	8.14	4.29	7.02	6.83	3.63
	2	7.12	4.33	6.03	6.13	2.97
42	1	8.11	5.67	7.32	7.23	4.96
	2	9.48	5.97	7.54	7.74	5.59
43	1	7.0	4.70	5.23	5.37	4.30
	2	7.35	4.50	5.25	6.24	4.25
44	1	6.52	4.91	6.37	6.26	3.57
	2	5.65	4.48	6.79	6.37	3.27
45	1	8.32	5.62	6.34	7.61	5.14
	2	9.18	6.20	7.30	6.90	4.57
46	1	4.56	2.95	3.50	3.67	1.83
	2	4.46	3.14	3.83	3.63	1.59
47	1	3.98	2.59	3.57	3.30	2.50
	2	4.46	2.87	3.53	3.67	2.46
48	1	15.18	4.32	8.90	12.07	1.89
	2	14.08	4.53	9.57	11.53	1.93

Note: * See footnotes on Table C.1

Table C.5 Modulus of Elasticity of Concrete in
the Main Laboratory Testing Program

Batch No.	Specimen No.	28-Day Moist Curing (x 10 ⁶ psi)		90-Day Moist Curing (x 10 ⁶ psi)	
		Compression	Flexure	Compression	Flexure
1	1	5.088	4.583	5.253	5.375
	2	5.010	4.733	5.302	4.841
2	1	5.078	5.083	5.058	5.563
	2	5.496	4.938	5.253	5.438
3	1	5.156	4.50	5.545	5.00
	2	5.204	4.688	5.253	5.344
4	1	5.350	5.034	5.088	6.334
	2	5.156	4.417	5.302	5.775
5	1	5.837	6.063	6.135	5.825
	2	6.274	6.536	6.503	5.749
6	1	5.272	4.734	5.494	6.108
	2	5.593	5.438	5.809	5.198
7	1	5.855	5.678	5.856	5.825
	2	5.764	5.763	5.856	6.186
8	1	5.107	5.355	5.611	5.333
	2	5.156	5.375	5.837	5.553
9	1	6.524	6.875	7.700	7.027
	2	6.347	6.313	7.314	6.804
10	1	5.584	5.546	5.945	6.00
	2	5.873	6.00	5.855	5.297
11	1	5.90	5.68	6.035	5.770
	2	5.629	5.622	5.855	5.709
12	1	5.647	5.070	5.646	5.113
	2	5.404	4.791	6.014	5.218
13	1	8.120	7.405	8.054	6.736
	2	7.782	7.136	8.180	8.165

Table C.5--continued

Batch No.	Specimen No.	28-Day Moist Curing (x 10 ⁶ psi)		90-Day Moist Curing (x 10 ⁶ psi)	
		Compression	Flexure	Compression	Flexure
14	1	5.359	5.488	5.744	5.709
	2	5.404	5.795	5.511	5.394
15	1	7.242	7.081	7.912	7.191
	2	7.440	7.308	7.912	7.284
16	1	5.128	4.645	5.708	5.00
	2	5.168	5.097	5.371	4.831
17	1	5.477	5.304	5.613	5.743
	2	5.731	5.231	5.674	6.268
18	1	6.935	5.608	7.058	7.070
	2	6.935	6.036	6.443	7.129
19	1	4.756	4.694	4.833	4.765
	2	4.637	4.942	4.805	5.294
20	1	5.619	5.458	5.699	6.471
	2	5.701	5.039	5.617	5.569
21	1	7.493	7.745	7.756	7.555
	2	7.646	6.727	7.506	8.021
22	1	4.968	5.116	5.564	5.131
	2	4.995	5.162	5.359	5.064
23	1	6.323	6.522	6.181	6.238
	2	6.515	5.382	6.278	6.941
24	1	5.618	5.360	5.843	5.490
	2	5.468	6.013	5.955	5.752
25	1	5.890	5.708	5.738	6.078
	2	5.916	5.701	5.764	5.838
26	1	5.226	4.690	4.965	4.379
	2	5.002	4.673	4.942	4.749

Table C.5--continued

Batch No.	Specimen No.	28-Day Moist Curing (x 10 ⁶ psi)		90-Day Moist Curing (x 10 ⁶ psi)	
		Compression	Flexure	Compression	Flexure
27	1	7.00	5.577	7.516	6.914
	2	6.33	6.407	7.884	6.00
28	1	6.160	5.048	6.237	5.545
	2	6.105	5.314	5.994	6.214
29	1	5.207	4.762	5.70	5.25
	2	5.119	5.264	5.531	5.521
30	1	6.405	5.977	6.366	7.865
	2	6.249	5.859	6.384	7.820
31	1	5.315	5.333	5.660	5.217
	2	5.465	5.068	5.578	5.169
32	1	5.560	5.342	5.691	6.087
	2	5.472	5.978	5.750	6.023
33	1	7.222	6.779	7.326	7.910
	2	7.119	7.523	7.601	8.063
34	1	5.744	6.763	5.797	5.926
	2	5.655	5.655	5.767	6.538
35	1	5.997	5.574	5.827	6.771
	2	6.019	6.474	5.917	6.181
36	1	5.514	5.833	5.625	5.811
	2	5.346	5.885	5.699	6.331
37	1	7.518	8.198	8.018	7.941
	2	7.954	7.529	8.057	7.964
38	1	7.601	8.171	8.031	8.198
	2	7.680	8.897	7.526	8.180
39	1	5.124	5.411	5.274	5.290
	2	5.397	6.248	5.289	6.581

Table C.5--continued

Batch No.	Specimen No.	28-Day Moist Curing (x 10 ⁶ psi)		90-Day Moist Curing (x 10 ⁶ psi)	
		Compression	Flexure	Compression	Flexure
40	1	7.562	8.919	7.386	7.914
	2	7.641	7.973	7.782	8.757
41	1	5.503	6.293	5.720	5.884
	2	5.699	5.957	5.750	5.581
42	1	5.389	4.919	5.298	5.923
	2	5.250	5.642	5.575	5.310
43	1	5.601	6.474	5.823	5.935
	2	5.568	6.667	5.766	6.045
44	1	5.186	6.019	5.708	6.968
	2	5.251	6.188	5.329	6.503
45	1	5.009	5.806	5.178	6.271
	2	5.058	5.714	5.028	5.752
46	1	5.251	5.478	5.908	5.323
	2	5.383	5.857	5.612	5.871
47	1	6.039	6.00	5.846	6.310
	2	5.809	5.955	5.870	5.335
48	1	4.902	5.104	5.551	6.108
	2	4.854	5.00	5.327	5.875

Table C.6 Mean Moduli of Rupture of Concrete at Early Age

Batch No.	Age* (hours)	Modulus of Rupture (psi)	Batch No.	Age* (hours)	Modulus of Rupture (psi)
1	19 1/2	414	25	28	679
2	20	584	26	26	398
3	46	605	27	25	679
4	21	578	28	24	631
5	-	-	29	27	545
6	50	581	30	25	744
7	21	552	31	26	496
8	19	337	32	26	524
9	22	572	33	25	656
10	72	614	34	24	573
11	22	610	35	24	533
12	22	596	36	25	661
13	23	715	37	24	857
14	25	695	38	24	626
15	28	777	39	25	507
16	30	568	40	26	682
17	24	599	41	24	658
18	22	604	42	24	601
19	28	365	43	24	586
20	24	580	44	25	656
21	24	740	45	24	609
22	27	626	46	25	625
23	25	711	47	27	664
24	24	443	48	26	456

Note: * Specimens were cured in the forms

Table C.7 Schmidt Hammer Rebound Number from Beam Specimen
in the Main Laboratory Testing Program

Batch No.	Specimen No.	Curing Condition*					
		1	2	3	4	5	6
1	1	30	29	35	34	33	-
	2	27	31	34	34	33	-
2	1	31	35	38	37	38	-
	2	32	32	38	35	36	-
3	1	34	34	37	39	38	-
	2	34	33	39	39	35	-
4	1	30	32	41	37	38	-
	2	28	33	39	38	36	-
5	1	42	45	43	43	42	-
	2	43	41	44	41	41	-
6	1	34	32	-	-	-	42
	2	34	33	-	-	-	41
7	1	37	39	-	-	-	44
	2	36	37	-	-	-	43
8	1	32	-	-	-	-	41
	2	33	-	-	-	-	41
9	1	-	-	-	-	-	42
	2	-	-	-	-	-	42
10	1	-	-	-	-	-	44
	2	-	-	-	-	-	43
11	1	-	-	-	-	-	44
	2	-	-	-	-	-	44
12	1	-	-	-	-	-	42
	2	-	-	-	-	-	43
13	1	-	-	42	41	41	46
	2	-	-	42	41	41	47

Table C.7--continued

Batch No.	Specimen No.	Curing Condition*					
		1	2	3	4	5	6
14	1	-	-	41	38	38	43
	2	-	-	42	40	39	43
15	1	-	-	45	42	41	46
	2	-	-	45	41	42	46
16	1	33	38	38	35	35	40
	2	32	38	38	35	35	40
17	1	37	40	41	36	38	44
	2	37	40	42	36	36	46
18	1	36	39	42	40	41	45
	2	36	38	42	40	40	46
19	1	28	29	33	32	32	37
	2	28	29	33	32	31	38
20	1	38	38	43	41	40	46
	2	37	38	43	41	41	45
21	1	41	41	45	41	40	46
	2	41	41	44	41	41	46
22	1	39	40	41	41	42	44
	2	39	39	41	41	42	43
23	1	41	45	47	45	44	47
	2	42	45	46	44	44	47
24	1	37	37	40	38	37	44
	2	37	37	40	37	38	44
25	1	39	41	42	41	40	44
	2	38	41	42	41	41	44
26	1	28	29	34	33	31	41
	2	29	29	34	31	31	41

Table C.7--continued

Batch No.	Specimen No.	Curing Condition*					
		1	2	3	4	5	6
27	1	36	37	43	39	39	45
	2	36	38	43	38	39	45
28	1	41	41	45	43	42	46
	2	41	41	45	42	42	46
29	1	35	36	40	36	37	42
	2	35	35	40	37	37	41
30	1	44	45	48	45	44	50
	2	44	45	48	44	45	50
31	1	33	34	40	36	38	41
	2	32	34	40	38	37	42
32	1	31	36	40	36	36	44
	2	30	35	41	36	36	44
33	1	36	36	39	37	38	44
	2	37	36	41	38	38	44
34	1	36	40	44	41	40	44
	2	37	40	43	40	40	44
35	1	37	40	44	41	41	46
	2	37	40	45	41	41	46
36	1	40	42	44	42	42	45
	2	40	42	45	41	41	46
37	1	43	46	46	44	44	47
	2	43	46	46	44	44	48
38	1	40	42	44	42	40	49
	2	40	42	45	41	40	49
39	1	35	36	40	38	38	44
	2	35	36	40	38	38	43

Table C.7--continued

Batch No.	Specimen No.	Curing Condition*					
		1	2	3	4	5	6
40	1	39	40	42	40	39	45
	2	39	40	41	40	39	45
41	1	38	40	45	44	44	46
	2	38	40	46	44	44	46
42	1	34	38	44	41	40	45
	2	34	38	44	40	41	45
43	1	37	38	43	41	40	46
	2	38	38	43	41	41	47
44	1	37	39	44	42	42	45
	2	37	39	44	42	42	46
45	1	36	38	40	39	38	42
	2	36	37	40	39	38	42
46	1	37	40	42	38	38	45
	2	37	40	43	39	38	46
47	1	39	42	44	42	40	47
	2	39	42	44	41	40	47
48	1	32	34	35	33	33	42
	2	31	34	35	33	33	42

Note: * See footnotes on Table C.1

Table C.8 Schmidt Hammer Rebound Number from Compression
Cylinder in the Main Laboratory Testing Program

Batch No.	Specimen No.	Curing Condition*					
		1	2	3	4	5	6
1	1	29	30	35	32	30	-
	2	28	30	34	31	31	-
2	1	30	29	38	38	38	-
	2	30	32	37	38	38	-
3	1	29	32	36	35	34	-
	2	30	31	39	35	33	-
4	1	29	30	38	36	34	-
	2	30	31	40	36	34	-
5	1	41	44	-	41	42	-
	2	43	45	-	41	42	-
6	1	37	36	-	-	-	41
	2	35	36	-	-	-	42
7	1	35	35	-	-	-	41
	2	35	39	-	-	-	40
8	1	31	-	-	-	-	40
	2	31	-	-	-	-	40
9	1	-	-	-	-	-	40
	2	-	-	-	-	-	40
10	1	-	-	-	-	-	45
	2	-	-	-	-	-	44
11	1	-	-	-	-	-	42
	2	-	-	-	-	-	42
12	1	-	-	-	-	-	41
	2	-	-	-	-	-	41
13	1	-	-	42	38	38	44
	2	-	-	42	38	38	44

Table C.8--continued

Batch No.	Specimen No.	Curing Condition*					
		1	2	3	4	5	6
14	1	-	-	38	37	37	42
	2	-	-	38	37	38	41
15	1	-	-	43	39	39	44
	2	-	-	41	40	39	44
16	1	32	31	36	35	34	35
	2	31	32	36	34	34	36
17	1	35	35	39	39	37	44
	2	33	34	39	38	37	44
18	1	31	36	39	37	37	42
	2	32	37	39	37	37	41
19	1	26	27	32	31	31	35
	2	26	27	32	31	31	35
20	1	35	35	40	38	37	43
	2	34	35	39	38	37	43
21	1	41	40	43	41	41	45
	2	41	40	42	41	41	44
22	1	34	35	41	40	38	41
	2	34	35	41	38	40	41
23	1	42	39	43	41	41	44
	2	42	42	42	41	41	43
24	1	34	34	37	36	34	41
	2	34	34	37	36	35	41
25	1	37	38	39	38	38	40
	2	37	37	40	38	39	41
26	1	28	26	31	29	29	35
	2	28	26	30	29	28	35

Table C.8--continued

Batch No.	Specimen No.	Curing Condition*					
		1	2	3	4	5	6
27	1	35	35	38	37	37	43
	2	35	36	39	37	37	43
28	1	41	40	44	41	41	44
	2	42	39	44	42	41	45
29	1	30	31	36	34	34	38
	2	30	31	35	34	35	38
30	1	42	41	43	41	41	45
	2	41	41	43	41	41	45
31	1	31	32	35	33	33	37
	2	31	31	35	33	33	36
32	1	30	31	34	33	33	37
	2	30	31	34	33	33	37
33	1	30	33	38	36	34	41
	2	30	33	39	36	34	41
34	1	35	36	41	37	38	42
	2	35	35	41	39	38	42
35	1	35	37	40	37	36	43
	2	35	36	40	38	37	42
36	1	37	39	39	38	37	42
	2	36	38	39	38	38	42
37	1	41	41	44	43	43	46
	2	41	41	44	43	42	46
38	1	38	39	42	40	39	44
	2	38	39	41	40	39	44
39	1	29	30	36	33	33	39
	2	28	30	35	33	33	39

Table C.8--continued

Batch No.	Specimen No.	Curing Condition*					
		1	2	3	4	5	6
40	1	36	37	41	38	37	42
	2	36	37	40	38	38	43
41	1	36	37	41	38	38	43
	2	36	37	41	38	39	43
42	1	32	34	39	37	37	41
	2	30	34	39	37	38	41
43	1	34	35	40	37	38	41
	2	34	35	40	37	38	42
44	1	36	37	39	38	38	41
	2	35	37	39	38	39	40
45	1	31	34	38	37	37	39
	2	32	34	38	37	37	39
46	1	35	36	40	39	39	41
	2	34	36	41	39	39	42
47	1	37	38	39	39	39	43
	2	38	38	39	40	39	43
48	1	27	28	33	33	32	37
	2	27	28	34	32	32	37

Note: * See footnotes on Table C.1

Table C.9 Schmidt Hammer Rebound Number from Splitting Tensile Cylinder in the Main Laboratory Testing Program

Batch No.	Specimen No.	Curing Condition*					
		1	2	3	4	5	6
1	1	28	27	30	29	27	-
	2	26	26	32	28	28	-
2	1	25	27	36	33	31	-
	2	27	26	35	33	31	-
3	1	26	26	31	30	30	-
	2	25	27	32	30	31	-
4	1	28	29	35	32	31	-
	2	27	30	34	31	27	-
5	1	38	36	-	-	-	-
	2	38	33	-	-	-	-
6	1	31	32	-	-	-	35
	2	32	32	-	-	-	33
7	1	30	32	-	-	-	36
	2	32	34	-	-	-	36
8	1	31	-	-	-	-	34
	2	32	-	-	-	-	34
9	1	-	-	-	-	-	35
	2	-	-	-	-	-	36
10	1	-	-	-	-	-	38
	2	-	-	-	-	-	38
11	1	-	-	-	-	-	36
	2	-	-	-	-	-	35
12	1	-	-	-	-	-	38
	2	-	-	-	-	-	37
13	1	-	-	35	34	33	38
	2	-	-	36	33	34	37

Table C.9--continued

Batch No.	Specimen No.	Curing Condition*					
		1	2	3	4	5	6
14	1	-	-	33	31	32	37
	2	-	-	35	33	33	37
15	1	-	-	36	34	35	39
	2	-	-	36	35	35	38
16	1	29	29	30	29	28	33
	2	29	29	29	29	29	32
17	1	32	33	35	33	32	37
	2	31	32	34	32	32	38
18	1	31	32	35	36	35	38
	2	30	33	36	35	34	37
19	1	23	23	29	28	28	30
	2	22	23	29	28	27	30
20	1	32	32	36	34	34	38
	2	32	31	35	34	34	38
21	1	34	34	39	37	36	39
	2	34	34	39	37	37	39
22	1	31	32	34	34	33	36
	2	31	32	33	33	33	35
23	1	35	37	38	37	37	39
	2	36	37	37	37	37	39
24	1	30	30	35	34	34	37
	2	30	30	35	34	35	37
25	1	35	34	37	35	35	37
	2	35	35	37	35	35	37
26	1	26	25	29	27	27	32
	2	26	25	29	27	27	32

Table C.9--continued

Batch No.	Specimen No.	Curing Condition*					
		1	2	3	4	5	6
27	1	32	34	35	34	34	37
	2	31	34	35	34	34	37
28	1	37	37	38	38	38	40
	2	37	37	38	37	38	40
29	1	27	26	32	30	30	35
	2	27	26	32	30	29	36
30	1	37	37	39	36	37	42
	2	38	38	39	37	38	41
31	1	28	28	33	30	29	34
	2	28	28	32	31	30	34
32	1	27	29	34	31	30	36
	2	27	28	34	31	32	36
33	1	28	31	34	34	33	36
	2	28	31	35	34	33	35
34	1	33	35	37	35	35	38
	2	33	34	37	35	35	38
35	1	32	32	37	35	34	39
	2	32	32	37	35	34	38
36	1	32	32	35	34	34	37
	2	33	31	35	34	34	37
37	1	36	36	38	37	38	40
	2	36	38	38	37	37	40
38	1	33	35	39	37	37	40
	2	34	35	39	37	37	40
39	1	26	27	32	28	28	36
	2	26	28	33	29	28	36

Table C.9--continued

Batch No.	Specimen No.	Curing Condition*					
		1	2	3	4	5	6
40	1	34	34	35	33	33	38
	2	34	34	35	33	33	38
41	1	32	33	37	35	35	39
	2	32	32	36	35	36	38
42	1	28	29	34	33	33	38
	2	28	29	34	33	33	38
43	1	32	32	36	34	34	39
	2	31	32	36	35	34	39
44	1	31	33	36	35	35	39
	2	31	33	36	35	35	39
45	1	29	29	36	34	33	37
	2	28	30	36	34	33	37
46	1	30	32	37	35	35	40
	2	31	32	38	34	35	41
47	1	35	36	37	36	35	40
	2	35	36	38	36	36	40
48	1	25	25	32	29	30	34
	2	25	26	32	31	31	34

Note: * See footnotes on Table C.1

Table C.10 Results of Tests on Silica Fume Concrete
at 28-Day Moist Curing

Batch No.	Percent Silica Fume	Specimen No.	Compressive Strength (psi)	Splitting Tensile Strength (psi)	Modulus of Rupture (psi)	Coefficient of Permeability ($\times 10^{-12}$ in/sec)
S1	5	1	8992	686	720	1.96
		2	9239	674	737	1.84
S2	5	1	8879	618	785	2.35
		2	9169	652	722	2.11
S3	10	1	9522	589	829	1.68
		2	9625	603	767	1.76
S4	10	1	9215	614	722	1.41
		2	9126	567	780	1.53
S5	15	1	9165	579	691	2.17
		2	9264	629	705	2.15
S6	15	1	9144	636	799	1.75
		2	9225	716	817	1.45
S7	20	1	9285	641	787	2.12
		2	9155	613	726	1.59
S8	20	1	9091	651	862	1.30
		2	9063	610	831	1.41
S9	10	1	9459	732	1401	1.23
		2	9791	836	1364	1.54
S10	20	1	9331	640	1308	1.23
		2	8974	670	1251	1.68

Table C.11 Results of Tests on Silica Fume Concrete
at 90-Day Moist Curing

Batch No.	Percent Silica Fume	Specimen No.	Compressive Strength (psi)	Splitting Tensile Strength (psi)	Modulus of Rupture (psi)	Coefficient of Permeability ($\times 10^{-12}$ in/sec)
S1	5	1	9522	691	855	1.87
		2	9809	670	828	1.79
S2	5	1	9866	710	884	2.23
		2	9317	664	828	2.04
S3	10	1	9851	725	817	1.61
		2	9866	754	884	1.73
S4	10	1	9756	714	849	1.62
		2	9820	751	958	1.63
S5	15	1	9568	632	882	1.78
		2	9607	644	815	1.72
S6	15	1	9455	640	994	1.52
		2	9526	713	905	1.77
S7	20	1	9356	633	944	1.02
		2	9565	686	963	1.66
S8	20	1	9225	642	977	1.63
		2	9660	653	955	1.49
S9	10	1	9692	704	1259	1.20
		2	9522	788	1183	0.83
S10	20	1	8765	613	1326	1.82
		2	9462	592	1272	1.58

Table C.12 Modulus of Elasticity of Silica Fume Concrete

Batch No.	Specimen No.	28-Day Moist Curing (x 10 ⁶ psi)		90-Day Moist Curing (x 10 ⁶ psi)	
		Compression	Flexure	Compression	Flexure
S1	1	5.813	5.169	5.908	6.318
	2	5.854	4.879	5.662	5.440
S2	1	5.766	5.324	5.711	5.903
	2	5.737	6.071	5.783	6.548
S3	1	5.454	5.990	5.615	5.649
	2	5.513	6.055	5.711	6.944
S4	1	5.472	5.932	5.612	5.473
	2	5.454	6.047	5.744	6.588
S5	1	5.295	5.064	5.232	5.729
	2	5.413	5.035	5.429	5.440
S6	1	5.295	6.481	5.455	6.506
	2	5.443	6.055	5.776	5.642
S7	1	5.257	4.745	5.380	4.892
	2	5.208	5.237	5.587	4.973
S8	1	5.232	5.671	5.331	5.878
	2	5.286	6.283	5.552	5.676
S9	1	6.958	8.062	7.075	8.216
	2	7.365	8.080	7.310	7.808
S10	1	7.153	7.654	6.868	9.348
	2	7.127	7.880	6.653	8.053

Table C.13 Schmidt Hammer Rebound Number from Silica Fume Concrete at 28-Day Moist Curing

Batch No.	Percent Silica Fume	Specimen No.	Compression Cylinder	Splitting Tensile Cylinder	Beam Specimen
S1	5	1	41	36	42
		2	41	36	42
S2	5	1	39	36	43
		2	39	37	43
S3	10	1	40	37	43
		2	40	36	43
S4	10	1	39	37	45
		2	39	37	45
S5	15	1	40	36	41
		2	40	36	41
S6	15	1	40	36	45
		2	40	37	44
S7	20	1	41	34	44
		2	40	34	43
S8	20	1	38	36	44
		2	39	36	43
S9	10	1	40	36	44
		2	40	36	44
S10	20	1	41	36	46
		2	41	37	46

Table C.14 Schmidt Hammer Rebound Number from Silica Fume Concrete at 90-Day Moist Curing

Batch No.	Percent Silica Fume	Specimen No.	Compression Cylinder	Splitting Tensile Cylinder	Beam Specimen
S1	5	1	42	37	46
		2	43	37	46
S2	5	1	41	38	45
		2	41	38	44
S3	10	1	42	37	44
		2	41	37	45
S4	10	1	43	37	46
		2	42	38	46
S5	15	1	41	37	44
		2	41	37	44
S6	15	1	42	37	46
		2	42	37	45
S7	20	1	42	36	45
		2	42	36	46
S8	20	1	41	36	46
		2	41	36	46
S9	10	1	39	38	43
		2	39	38	43
S10	20	1	42	38	47
		2	43	37	47

Table C.15 Results of Tests on Field Concrete

Project No.	Curing Condition*	Specimen No.	Compressive Strength (psi)	Splitting Tensile Strength (psi)	Modulus of Rupture (psi)	Coefficient of Permeability ($\times 10^{-11}$ in/sec)
1	7-MC	1	5508	465	628	-
		2	5550	513	657	-
	28-MC	1	6572	515	652	4.82
		2	6367	507	663	5.08
	FIELD	1	6760	457	-	6.69
		2	6425	423	-	6.99
	90-MC	1	7031	553	657	1.18
		2	6788	529	672	1.35
2	7-MC	1	5600	544	627	-
		2	5285	480	587	-
	28-MC	1	6434	636	640	1.03
		2	5971	601	697	1.06
	FIELD	1	6553	589	-	2.79
		2	5804	584	-	2.39
	90-MC	1	7092	569	708	0.95
		2	7959	594	678	1.09
3	7-MC	1	4351	350	549	-
		2	4064	357	572	-
	28-MC	1	4705	503	591	7.81
		2	4620	499	562	7.11
	FIELD	1	5454	531	-	11.90
		2	5581	510	-	10.65
	90-MC	1	4984	539	626	6.42
		2	5140	497	599	7.93

Table C.15--continued

Project No.	Curing Condition*	Specimen No.	Compressive Strength (psi)	Splitting Tensile Strength (psi)	Modulus of Rupture (psi)	Coefficient of Permeability ($\times 10^{-11}$ in/sec)
4	7-MC	1	4507	382	562	-
		2	4238	341	503	-
	28-MC	1	5313	494	477	7.28
		2	4995	499	524	8.71
	FIELD	1	5955	512	-	4.47
		2	6306	458	-	4.00
	90-MC	1	5267	525	567	8.03
		2	5384	550	534	6.30
5	7-MC	1	4439	446	580	-
		2	4255	312	643	-
	28-MC	1	5150	412	568	5.42
		2	5097	369	534	5.80
	FIELD	1	4172	440	-	11.05
		2	3535	466	-	9.77
	90-MC	1	5865	541	572	4.48
		2	5815	537	568	5.68
6	7-MC	1	5359	323	518	-
		2	4949	446	584	-
	28-MC	1	6424	452	555	4.81
		2	6604	455	624	4.15
	FIELD	1	6083	448	-	5.93
		2	6537	460	-	6.52
	90-MC	1	7329	598	590	2.74
		2	6986	567	609	2.87

Note: * Curing conditions:

7-MC = 7-day moist curing

28-MC = 28-day moist curing

FIELD = cored samples from jobsite at 28-day after placement

90-MC = 90-day moist curing

- Cored samples are 4 inches ϕ x 8 inches

Table C.16 Modulus of Elasticity of Field Concrete

Batch No.	Specimen No.	28-Day Moist Curing (x 10 ⁶ psi)		90-Day Moist Curing (x 10 ⁶ psi)	
		Compression	Flexure	Compression	Flexure
1	1	4.014	4.541	4.140	4.785
	2	4.064	4.527	4.077	4.986
2	1	5.246	4.734	5.463	4.742
	2	5.026	4.855	5.906	4.736
3	1	4.005	3.934	3.807	4.459
	2	4.058	3.923	3.851	4.345
4	1	4.157	4.086	4.369	4.369
	2	4.096	4.234	4.306	4.977
5	1	3.676	3.711	4.190	4.124
	2	3.763	4.115	4.067	4.167
6	1	4.168	4.486	4.506	4.626
	2	4.339	4.331	4.511	4.729

Table C.17 Schmidt Hammer Rebound Number from Field Concrete

Project No.	Percent Silica Fume	Specimen No.	Compression Cylinder	Splitting Tensile Cylinder	Beam Specimen
1	7-MC	1	27	26	35
		2	27	26	35
	28-MC	1	34	28	39
		2	33	28	39
	FIELD	1	35	-	-
		2	35	-	-
	90-MC	1	34	30	40
		2	34	30	40
2	7-MC	1	27	26	32
		2	26	26	31
	28-MC	1	29	27	33
		2	28	27	33
	FIELD	1	-	-	-
		2	-	-	-
	90-MC	1	35	32	36
		2	37	31	35
3	7-MC	1	25	21	26
		2	24	22	25
	28-MC	1	26	23	28
		2	26	23	29
	FIELD	1	33	-	-
		2	34	-	-
	90-MC	1	27	24	31
		2	27	24	31

Table C.17--continued

Project No.	Percent Silica Fume	Specimen No.	Compression Cylinder	Splitting Tensile Cylinder	Beam Specimen
4	7-MC	1	24	23	26
		2	24	23	26
	28-MC	1	26	25	28
		2	27	26	28
	FIELD	1	33	-	-
		2	33	-	-
	90-MC	1	27	26	30
		2	27	27	30
5	7-MC	1	25	23	27
		2	25	22	27
	28-MC	1	28	27	30
		2	28	25	30
	FIELD	1	32	-	-
		2	31	-	-
	90-MC	1	29	27	32
		2	30	27	32
6	7-MC	1	30	26	31
		2	28	28	32
	28-MC	1	31	28	34
		2	31	28	34
	FIELD	1	35	-	-
		2	36	-	-
	90-MC	1	33	31	34
		2	32	31	34

Note: * See footnotes on Table C.16

APPENDIX D
SUMMARY OF BURR-FOSTER Q-TEST RESULTS

Table D.1 Summary of BURR-FOSTER Q-Test Results

Experimental Design No.	Variable*	Degree of Freedom	No. of Samples	Q Statistics	Homogeneity** of Variance
1	fr	1	108	0.0292	Accepted
	f'_C	1	108	0.0420	Accepted
	fct	1	108	0.0236	Accepted
	K	1	90	0.0414	Accepted
2	fr	1	108	0.0304	Accepted
	f'_C	1	108	0.0290	Accepted
	fct	1	108	0.0246	Accepted
	K	1	90	0.0414	Accepted
3	fr	1	108	0.0284	Accepted
	f'_C	1	108	0.0262	Accepted
	fct	1	108	0.0259	Accepted
	K	1	90	0.0509	Accepted
4	fr	1	72	0.0655	Accepted
	f'_C	1	72	0.0452	Accepted
	fct	1	72	0.0368	Accepted
	K	1	60	0.1014	Accepted
5	fr	1	36	0.0621	Accepted
	f'_C	1	36	0.0782	Accepted
	fct	1	36	0.0825	Accepted
	K	1	30	0.0675	Accepted
6	fr	1	20	0.1109	Accepted
	f'_C	1	20	0.1668	Accepted
	fct	1	20	0.0990	Accepted
	K	1	20	0.1463	Accepted

Note: * Variables:

fr = Modulus of rupture

f'_C = Compressive strength

fct = Splitting tensile strength

K = Coefficient of permeability

** $\alpha = 0.001$

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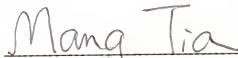
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BIOGRAPHICAL SKETCH

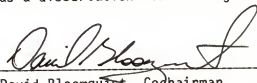
Prasit Soongswang was born in Nan, Thailand, on May 8, 1957. In March 1979, he obtained a bachelor's degree in engineering from Chulalongkorn University, Bangkok, Thailand. He worked as a structural engineer at R. K. V. Engineering Consultant Company, Ltd., Bangkok, Thailand, for four and one-half years. In January 1984, he began studies towards a master's degree in civil engineering at the Polytechnic Institute of New York, Brooklyn, New York. He received a Master of Science degree in civil engineering in January 1985. He worked with Lev Zetlin Associates, Inc., Engineers and Designers, New York, New York, for one and one-half years. In August 1986, he entered the University of Florida to pursue his studies for a Doctor of Philosophy degree in civil engineering. He received the University of Florida President's Recognition Award in March 1989.

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Mang Tia, Chairman
Associate Professor of Civil Engineering

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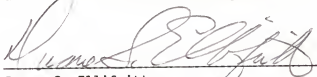
David Bloomquist, Cochairman
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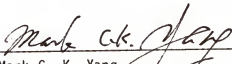
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Professor of Civil Engineering

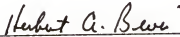
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Mark C. K. Yang
Professor of Statistics

This dissertation was submitted to the Graduate Faculty of the College of Engineering and to the Graduate School and was accepted as partial fulfillment of the requirements for the degree of Doctor of Philosophy.

August 1989



Dean, College of Engineering

Dean, Graduate School